MINE SURVEYING AND LEVELLING

A GUIDE TO MINE SURVEYING

REVISED AND ENLARGED SIXTH EDITION

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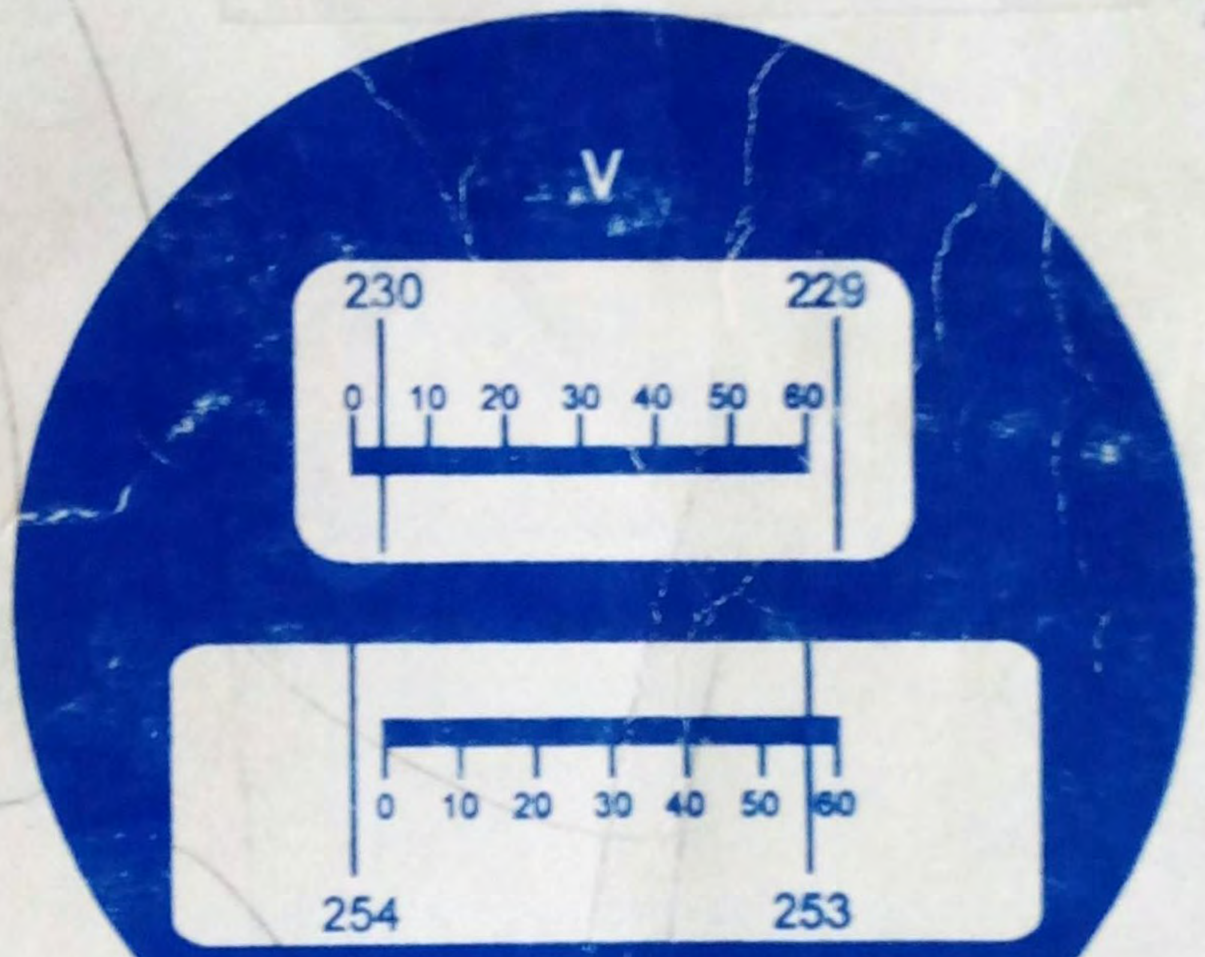

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CHAPTER I

THEODOLITE SURVEYING

THEODOLITES

The theodolite is the most precise instrument designed for the measurement of horizontal as well as vertical angles with considerable accuracy and it has a wide field of usefulness in surveying such as laying off horizontal angles, locating points on line, prolonging survey lines, establishing grades, determining difference in elevation, setting out curves, etc.

The theodolite was invented by Jonathan Sisson, an instrument maker in London, in 1720, Later on Jasse Ramsden improved the design by introducing three foot geodetic theodolite in about 1775. At one time the vernier and micrometer microscope theodolites were most common, but these are being superseded by instruments of the optical-micrometer type.

CLASSIFICATION OF THEODOLITES

Theodolite may be classified as -

1. Transit theodolite.
2. Non-transit theodolite.

A *transit theodolite* (or simply *transit*) is one in which its telescope can be revolved or transited through complete revolution about its horizontal axis in the vertical plane.

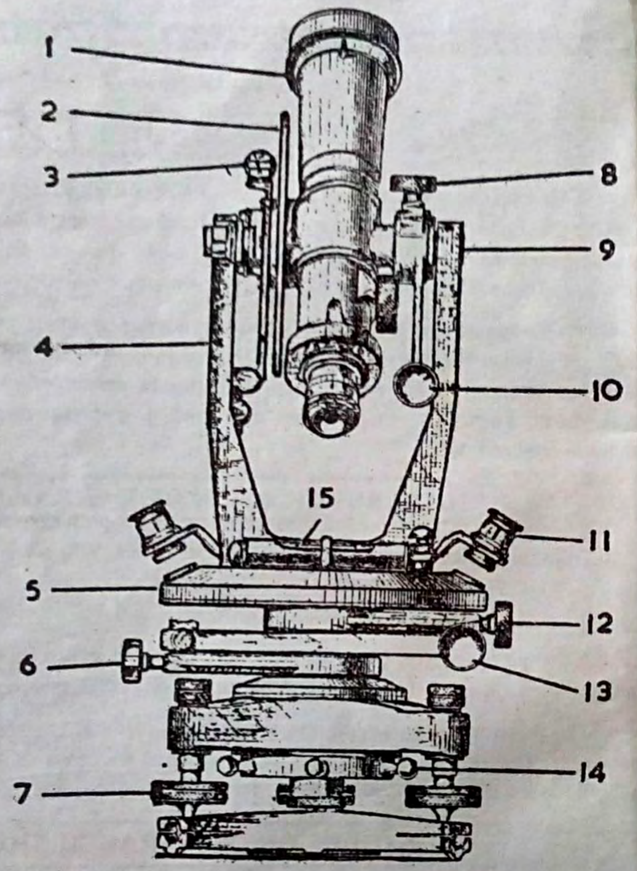
The theodolite in which the telescope can not be transited is called *non-transit theodolite*. The non-transit theodolite may either be plain or Y-type theodolite. The transit theodolites are largely used now-a-days and non-transit types have become obsolete now.

CONSTRUCTION OF A TRANSIT THEODOLITE

A standard transit theodolite (vernier type) consists of a telescope mounted on a *horizontal axis* so that the telescope can be rotated in a vertical plane. The horizontal axis is supported on two *vertical standards* and the telescope with standards can be rotated in a horizontal plane. The standards are attached to the inner spindle of the vertical axis by means of the vernier arm of the horizontal circle, and the two verniers for the horizontal circle are usually

marked A and B. The outer spindle of the vertical axis carries the *horizontal circle* and the outer spindle may itself revolve in the outer socket which is attached to the *levelling head*. Each of these spindles may be fixed in any desired position by means of clamps. The top clamp is called the *vernier clamp*, and it holds the vernier to the graduated circle, while the bottom or *body clamp* holds the graduated horizontal plate to the body of the instrument. Both clamps are fitted with slow-motion or *tangent screws*.

1. Telescope.
2. Vertical circle.
3. Split level on vertical index plate.
4. Vertical standards.
5. Horizontal circle.
6. Body clamp.
7. Levelling screws.
8. Vertical circle clamp.
9. Horizontal axis.
10. Vertical circle tangent screw.
11. Reading microscope
12. Vernier clamp.
13. Slow-motion or Tangent screw.
14. Quick centring device
15. Spirit level on horizontal plate.



Vernier Theodolite

The *vertical circle* is fixed to the telescope and readings are obtained from two verniers, C and D, attached to the end of a vernier arm which is fixed to one of the vertical standards. The horizontal circle is graduated in degrees and thirds of a degree from 0° to 360° and figured in a clockwise direction. The graduations are cut on silver rings recessed into bevelled edges of the

circles and two verniers, 180° apart, are fitted with *reading microscope* of the Ramsden type with screw focusing adjustment and light diffusers to ensure uniform illumination of the graduated circle and vernier.

Telescope on modern theodolites are usually of the internal focusing type. Spirit levels are fitted to the horizontal and vertical index plates, and an additional level is sometimes fitted to the telescope. Three screw levelling bases are usual and a quick centring device may be fitted to the instrument if desired.

Mining Theodolite should be constructed to exclude dust or damp as completely as possible, the overall dimensions of the instrument should be as small as possible, a centring device should be incorporated above the levelling base so that the theodolite can be set up under roof stations without having to re-level after accurate centring and the instrument should be constructed so that it can be manipulated with the minimum amount of movement on the part of the surveyor.

The following particulars refer to the specification of mining theodolite (Rand Model manufactured by cooke, Troghton and Simms, Limited) fulfilling the above conditions :

Telescope : Internal focusing type 38 mm aperture X 25 magnification, 210 mm focal length, field of view of telescope 1°41', stadia correction nil, nearest focusing distance 1.65 m, distance at which 0.003 m can resolve 300 m. Interchangeable glass diaphragms with stadia lines spaced 1 ; 100. Vertical angles can be observed down to 55° of depression without the use of auxiliary telescope.

Circles : Horizontal circle, 115 mm diameter, vertical circle, 100 mm diameter, both reading by verniers to 20" of arc. The circles are totally enclosed and the hinged glass covers over the verniers can be easily opened to remove dust or moisture on the underside.

Spirit Levels : Sensitivity in seconds of arc part 2 mm run are - Horizontal plate 40" to 45". Vertical index circle 15" to 20" and telescope 20" to 25".

Levelling Stage : Three-screw, with provision to take up wear, and thimbles covering the tops of the foot-screws to exclude dirt.

Centring Device : Fine centring arrangement with 20 mm movement situated above the foot-screws. Accurately placed reference marks on both side of transit axis for precise centring under roof marks.

Tripod : Made of duralumin or ash with telescopic legs clamped by wing nuts.

Accessories : Gunmetal plummet with silk cord, suspension hook, and grip washer. Camel hair brush for cleaning dust, screw-driver and adjustable tools. Diagonal eyepiece for steep rising sights. Auxiliary telescope for steep dipping sights. Colour prism reflector to illuminate the diaphragm in underground work. Tubular compass.

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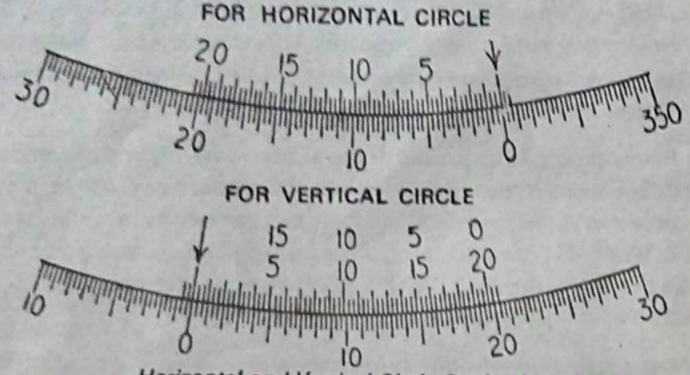
THE READING OF THE CIRCLES

The figure shows the graduations on the horizontal circle and a vernier sub-divided into 20" divisions, also the figuring of a vertical circle and vernier of the theodolite.



Horizontal and Vertical Circle Graduations

The main disadvantage of the vernier system of circle reading is that, in spite of specially attached magnifiers, it is not always easy to decide upon the graduations which are exactly coincident, particularly when the circle and vernier become tarnished.

Micrometer Microscope : The difficulties in reading of the circles by vernier system are to some extent overcome by the use of the micrometer microscope, which replaces the vernier. A special microscope magnifies a portion of the circle, which is read against a V shaped index notch. A double cross-hair lies in the field of view of the microscope and is so geared that one complete revolution of a micrometer drum, situated at the side of the microscope moves the cross-hair through exactly one division of the circle. The circle of theodolites of this type are usually graduated in intervals of 10 minutes and consequently, the micrometer drum is graduated into ten main divisions, each representing 1 minute, and the main divisions are subdivided into six parts, each representing 10 seconds. The circle can, therefore, read directly to the nearest 10 seconds, and by estimation to the nearest 2 seconds.

To read the circle the degrees and minutes are first read off against the index mark, the cross-hairs are then traversed until they bisect the lower circle-graduation, and micrometer drum is read. The sum of the coarse and fine readings give the actual circle reading.

Although the micrometer microscope relieves the observer of the strain of vernier reading, two readings are still necessary for each circle setting.

Optical Micrometer : The optical-micrometer-reading instrument provides for reading both horizontal and vertical circles in the same eye-piece and only one reading of each is registered. The method of reading is invariably clear and simple, the single readings reduce observation strain and time, and since the reading eye-piece is usually attached closely alongside the telescope, no movement of the observer around the instrument is necessary. All circles, auxiliary

scales, and reading microscopes are totally enclosed, and a single adjustable mirror is used to reflect light through both circle reading systems; either day light or artificial light can be used.

In the simpler instruments, the circle is actually read at one point only, special care being taken by manufacturers to ensure that it is accurately and permanently centred. Light is reflected through an adjustable mirror into a condenser which concentrates it on the point at which the circle is to be read. The illuminated circles are magnified by a system of lenses and their images directed, by means of fixed prism, through a movable parallel plate into the reading eye-piece. In the field of view of the latter, portions of both circles are seen in conjunction with micrometer scale, all three being magnified to suitable proportions by the eye-piece.

The micrometer scale is actually a portion of a graduated arc which is geared to a movable parallel plate, and both are controlled by an external micrometer drum. Rotation of the drum rocks the parallel plate which in turn displaces the images of the circles and allows the one nearest to the index mark to be made coincident with it. At the same time the arc is swung so that the reading of the micrometer scale in the eye-piece registers the angular distance through which the circle images have been displaced. The final reading is therefore the sum of the particular circle reading, which coincides with its index, and the index reading of the micrometer scale. Most instruments of this type have circles graduated in 20 minute divisions and the micrometer scale in 20 second divisions. It is thus quite easy to estimate the fine reading to the nearest 10 seconds.

For registration of a reading from one point on the circle only and to achieve work of the highest order theodolites have been designed which bring the images of two opposite portions of the circle simultaneous into the reading eye-piece where they are automatically meaned to give a single accurate reading.

OPTICAL THEODOLITES

There have been revolutionary change in the design of theodolites due to the reading arrangement made available by the introduction of transparent glass circles. The Introduction of transparent glass circle has resulted the following advantages :

1. Glass circles with opaque graduations have overcome the illumination problem.
2. Glass circles can now be obtained with diameters about half that of the older metal circles and yet with increased accuracy, therefore reducing the size and weight of the instruments without loss in accuracy.
3. Glass circles and auxiliary scales are divided with extreme accuracy, and as the graduations consist of opaque lines on a transparent medium, they appear as black lines against a white background. The circles can be easily and evenly illuminated, and since the graduations show up clearly they can be subjected to a high degree of magnification by powerful compound microscopes.
4. There is no loss of accuracy due to distortion or ageing since the optical glass is the most stable substance.
5. Neither the circles nor the auxiliary scales are liable to tarnish.

Short, high-powered, internal focussing telescopes are used on all modern optical theodolites and the short length of the telescope allows the height of the vertical standards to be reduced, thereby reducing the size and weight of the whole theodolite. In order to get readings smaller than the divisions on the circles auxiliary scales or graticules are introduced into the focal plane of the eye-pieces, where they are clearly defined and therefore easily read. The reading eye-pieces are normally observed from the same stance of the main telescope and this saves time and reduces the risk of disturbance which is incurred when the observer walks round the instrument.

All the circles, auxiliary scales, and reading microscopes are totally enclosed, and a single adjustable mirror is used to reflect light through both circle reading system; either daylight or artificial light can be used.

Most of the optical theodolites are provided with the centring arrangement by optical plummet system and the body of the instrument can be detached from the tribrach to enable the instrument to be interchanged precisely with targets on other tripods as in the three-tripod method of traversing thus employing the method of controlled centring.

WATTS MICROPTIC THEODOLITE : Messers Hilger and Watts Ltd. manufacture three models of microptic theodolites, No. 1, No. 2 and No. 5. Out of three, No. 2 is most precise having a least count of 1".

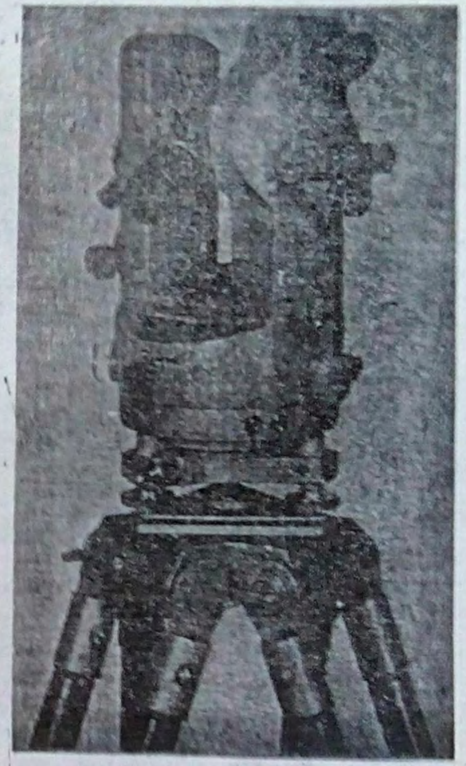
The optical system in watts No. 1 Microptic transit is indicated in Fig. It will be observed that light is reflected through the circles and also through an optical micrometer to the eye-piece of the circle reader which is situated on the outside of one of the vertical standards.

On looking through the eye-piece of the circle reader three illuminated apertures can be seen. By revolving the milled head controlling the optical micrometer a graduation mark on the horizontal circle is brought accurately to the triangular index mark. The circle graduation is then noted and the minutes and seconds can be read directly to 20" and estimated to the nearest 5" in the lower aperture. Operating the optical micrometer to bring a graduated mark on the vertical scale opposite the index mark allows the vertical angle to be read.

The magnification of the telescope is x 25, stadia correction nil, stadia interval 1 : 100. An optical plummet for ground section is a standard fitting. The main body of the instrument is made

of gunmetal which gives maximum durability under mining conditions. The weight of the instrument is 3.86 kg and the dimension of the metal carring case 34.3 x 19 x 18 cms.

TAVISTOCK THEODOLITE : The Tavistock Type II theodolite embodies a coincidence method of reading the circles, there is only one micrometer for both circles and a single reading eye-piece lies parallel with the main telescope. The general arrangement of the instrument is shown in Fig.



Both circles are graduated every 20 minutes and marked in degrees from 0° to 359° in a clockwise direction, the vertical circle indicating 0° at the zenith with the telescope in the face left position. Since there is only one micrometer, a change-over control is provided at the base of one of the vertical standards to enable the observer to select which circle is to be viewed. Both circles are illuminated by the one minor.

A coincidence method of reading is employed in this instrument which means that graduation marks diametrically opposite to each other are made to coincide by operating the micrometer setting screw and the reading obtained is therefore free from eccentricity errors. Coincidence takes place at intervals of 10 minutes and the micrometer indicates the minutes and seconds. The reading can be taken direct to one second of arc and be estimated to 0.25 or 0.5 second. If coincidence takes place opposite the reading-mark then the micrometer reading is simply added to the marked scale reading. When coincidence occurs symmetrically on either side of the reading-mark then 10 minutes must be added to the circle reading, short of the reading-mark, in addition to the micrometer reading.

The magnification of the telescope is x 25, stadia correction nil, stadia interval 1 : 100, minimum focussing distance 1.8 metre. An optical plummet is incorporated in the instrument. The weight of the instrument is about 5 kg and the dimension of the case 33 x 20.3 x 2.8 cms.

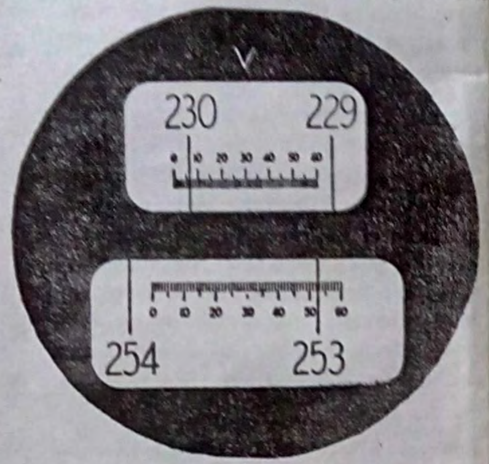
OPTICAL SCALE THEODOLITE : The cooke, Troughton and Simms, optical scale theodolite V 208 model is provided with a mirror for reflection of light into the instrument. Controlled by prisms and lenses the rays of light are passed through the horizontal and vertical glass circles and are guided into the eye-piece of the circle reading microscope which lies parallel with

the telescope. Both circles are marked in degrees and figured 0° to 359° in a clockwise direction, the figuring on the vertical circle indicating 0° at the zenith. An auxiliary scale, divided into 60 divisions, is seen in the field of view of the reading microscope and the length of this scale corresponds to the length of 1° on the circles so that the reading of angles in degrees and minutes can be observed directly as indicated in the Fig.

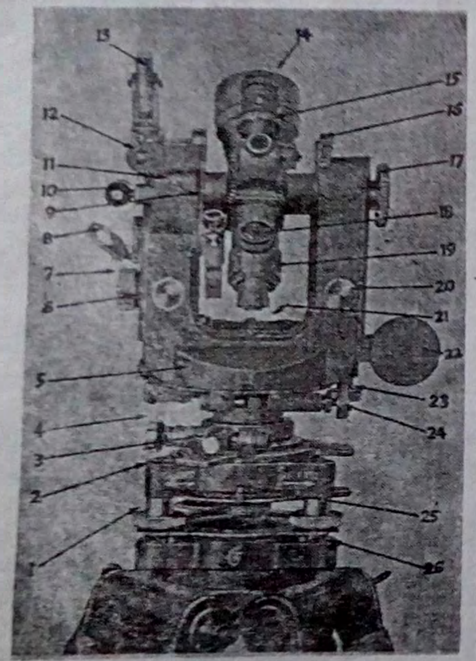
Vertical Circle = 230°06'30"

Horizontal Circle = 252°52'30"

The magnification of the telescope is x 21.5, stadia correction nil, stadia lines ratio 1 : 100, nearest focussing distance 1.4 metre. An optical plummet incorporated in the instrument for centring over a ground mark. The weight of the instrument is about 4 kg.



View in Reading Microscope of Optical Scale Theodolite



1. Foot screw. 2. Tribrach. 3. Clamp for lower plate. 4. Slow-motion screw for lower plate. 5. Casing of the horizontal circle. 6. Slow motion screw for the altitude spirit level. 7. Micrometer milled head. 8. Illumination reflector for circles. 9. Circle reading eyepiece. 10. Electric lamp for illumination of the circles and the altitude level. 11. Casing of the vertical circle. 12. Altitude spirit level. 13. Reversible mirror for altitude spirit level. 14. Object glass. 15. Optical plummet. 16. Facing for magnetic compass. 17. Focussing head for the telescope. 18. Screw focussing eye-piece. 19. Cover to the reticule adjustment screws. 20. Slow motion for the telescope. 21. Plate spirit level. 22. Battery for electric illumination. 23. Clamp for the telescope. 24. Slow-motion screw for the upper plate. 25. Clamp for centring motion. 26. Trivet stage.

DEFINITIONS AND TERMS

1. **The Vertical Axis :** The vertical axis is the axis about which the instrument can be rotated in a horizontal plane. This is the axis about which the lower and upper plates rotate.
2. **The Horizontal Axis :** The horizontal or trunnion axis is the axis about which the telescope and the vertical circle rotate in vertical plane. It is also known as transverse axis.
3. **The Axis of Level Tube :** The axis of the level tube or the bubble line is a straight line tangential to the longitudinal curve of the level tube at its centre. The axis of the level tube is horizontal when the bubble is in the centre.
4. **The Axis of Telescope :** It is the imaginary line joining the optical centre of the object glass and the centre of the eye-piece.
5. **The Line of Sight or Line of Collimation :** The imaginary line passing through the intersection of the horizontal and vertical cross-hairs and the optical centre of the object glass and its continuation is called the line of sight or line of collimation.
6. **Centring :** The process of setting the theodolite over or under a station mark is known as centring.
7. **Transiting :** It is the process of rotating the telescope over its supporting axis, (horizontal axis) through 180° in a vertical plane. Since the line of sight is reversed in this operation, it is also known as reversing or plunging. The limb and vernier plate remaining stationary in this process.
8. **Swinging the Telescope :** The process of turning the telescope about the vertical axis of the instrument in the horizontal plane is called swinging. The swing may be right or left swing according to the rotation of the telescope in a clockwise or anti-clockwise direction.
9. **Telescope Normal :** A telescope is said to be normal or direct when the face of the vertical circle is to the left and the bubble of the telescope up.
10. **Telescope Inverted :** A telescope is said to be inverted or reversed when the face of the vertical circle is to the right and the bubble down.
11. **Face Left Observation :** It is an observation of an angle (horizontal or vertical) made with the face of the vertical circle on the left of the observer.
12. **Face Right Observation :** If the face of the vertical circle is to the right of the observer, the observation is known face right observation.
13. **Changing Face :** It is the operation of bringing the face of the telescope from left to right and vice versa. It is carried out in two operations : firstly, the telescope is transited and then in the second operation to swing the telescope.
14. **Turning in Azimuth :** It means rotating the limb (body) together with the vernier-plate and all its attachments, in a horizontal plane; or in rotating only the vernier-plate and attachments, the limb remaining stationary.
15. **Turning in Altitude :** It means turning the telescope, together with the vertical circle, in a vertical plane, the limb and vernier-plate remaining stationary.

16. **Size of Theodolite :** The diameter of the graduated circle on the lower plate defines the size of the theodolite. 8 cm to 12 cm sized theodolites are used for general survey work, while 14 to 25 cm sized are used for triangulation work.

QUALITIES OF THE TELESCOPE

The qualities of the telescope are as follows –

1. **Aplanatism :** It means the absence of spherical aberration.

2. **Achromatism :** It means the absence of chromatic aberration.

3. **Definition :** It means the power of a telescope to produce a clear, distinct and sharp image. It depends upon the accuracy or grinding and centring and on the polishing of the lenses.

4. **Illumination :** It means the relative amount of light conveyed by the lenses of a telescope from a point to the eye compared with the light sent from the point to the naked eye. An object seen through a telescope will appear brighter in proportion to the amount of light passing from it to the eye. The brightness of the image depends on the aperture and quality of the lens, and it varies inversely as the square of the magnifying power. A high magnification will produce a dull image, while a lower magnification produces a bright image.

5. **Size of Field :** This is the whole circular area seen through the telescope. It is measured by the angle formed at the optical centre of the object glass by the diameter of the field of view. A wide field of view is desirable, since it facilitates the discovery of distant objects before bisecting them exactly. The angle varies from about 1^0 to 2^0. The angle does not depend upon the size of the object glass, but it increases as the size of the eye-piece increases and decreases as the distance between the objective and the eye-piece increases and also as the magnifying power increases.

Magnification : This is the ratio of the focal length of the objective to that of the eye-piece. A telescope with an objective of 25 cms and an eye-piece of 1.25 cm. focal lengths has a magnification of $\frac{25}{1.25} = 20$. This is slightly altered by the focusing of the eye-piece on to the diaphragm. The brightness of the image rapidly falls of with increase of magnifying power, and unnecessary power is sometimes a disadvantage. For transits the magnifying power is from 20 to 24 diameters, while for levels it is about 30 diameters. In precise instrument it is about 40 to 80 diameters.

ADJUSTMENT OF THEODOLITE

In order that a theodolite may be used accurately to measure horizontal and vertical angles, certain requirements must be fulfilled in regard to its general construction, the relative disposition of its component parts, and the manner in which it is set up.

Some defects are non-adjustable, these may be described as *instrumental imperfections*. Other defects are adjustable and they are divided into two classes, namely (a) *Permanent adjustment*, and (b) *Temporary adjustments*.

Instrumental Imperfection : These can not be adjusted, although they can be allowed for, and they include -

(a) Eccentricity of Verniers : The line joining the zeros of the verniers should be a truly diametral line through the axis of the spindle, and their reading should differ by exactly 180^0. If not, the verniers are eccentric. The error can be eliminated by reading both the verniers and taking the mean of the two.

(b) Eccentricity of Centres : The graduated and vernier-plates should be truly concentric (the centre of the graduated circle must coincide with the centre of the vernier plate), so that the difference between the two vernier readings remains constant in all the positions around the circle. If not, the error may be allowed for by reading both verniers and averaging the result.

(c) Graduation Errors : The graduation marks on the plates should be correctly and evenly spaced. This may usually be taken for granted in modern theodolites, but may be checked by observing a test angle on different parts of the plates. The effect of graduation errors may be reduced to any required extent by repeating the measurement on various parts of the horizontal circle and averaging.

PERMANENT ADJUSTMENT : Permanent adjustments are those required only at comparatively rare intervals, in order to detect and eliminate any defects that may have developed during use. In general five adjustments are possible, and these should be carried out in the order named –

1. **Plate Levels :** The axis of the plate spirit levels must be perpendicular to the vertical axis so that the plate can revolve in a horizontal plane about the vertical axis.

2. **Collimation Adjustment :** The line of sight or line of collimation must coincide with the optical axis of the telescope and must be at right angles to the horizontal (or transit, transverse, or trunnion) axis of the telescope so that when the telescope is revolved on its horizontal axis, the line of collimation generates a plane.

3. **Horizontal Axis :** The horizontal axis must be perpendicular to the vertical axis and be truly horizontal When adjustment 1 and 2 have been effected and the instrument has been levelled up.

4. **Telescope Level :** The axis of the telescope spirit level must be parallel to the line of sight.

5. **Vertical Circle or Index :** The reading of the verniers on the vertical circle should be zero when the line of sight is horizontal.

Of the foregoing five adjustments the first three are required for horizontal angles and the last two for vertical angles. The collimation adjustment is specially necessary when the instrument is used for traversing by continuous azimuth method, and reading with verniers on both faces of the instrument automatically corrects for collimation error, index error, eccentricity of verniers and eccentricity of centres. The horizontal axis adjustment is important if a theodolite traverse is made in steeply inclined roadways because reading with verniers on both faces corrects for horizontal axis error when measuring horizontal angle but not when measuring vertical angles, although it minimises the error.

TEMPORARY ADJUSTMENTS : Temporary adjustments are those required everytime the instrument is set over or under a station. These are –

1. Setting up the instrument and carefully plumbing over or under the station mark.
2. Levelling the instrument, and
3. Elimination of Parallax.

The adjustments are made in the following manner –

1. **Setting up :** The theodolite is placed on its stand over or under the station mark with a plumb-line suspended from a small hook at the base of the instrument in the case of a floor station or from the station point itself in the case of a roof station. To facilitate the latter, the telescope carries a spirit level to set the telescope horizontal and a centring thorn marking the instrument centre. The instrument is approximately centred and levelled by adjusting the tripod legs and the plumb-bob is exactly centred over or under the station by means of sliding plate or other centring device. Accurate plumbing, however, is very difficult in windy weather or in mine roads where the velocity of air current is high. Some modern instrument have an optical plummet which can be used for accurate centring.

2. **Levelling up :** This is carried out by manipulation of levelling screws with reference to the plate bubbles. The instrument is rotated horizontally, until one of the plate levels is parallel to two of the foot screws. By means of these screws the bubble of this level is brought to the centre of its run, and by the third screw the bubble of the other level is centred. If the bubble of the levels is still retain their centres, on rotating the instrument horizontally through 180°, the adjustment is effected. If not, the process is repeated until finally both bubbles are centred.

3. **Ellimination of Parallax :** When the image of the object viewed, formed by the object glass, fails either short of, or beyond the place of the cross hairs of diaphragm, the errors arising from this is known as parallax. It's existence is determined by moving the eye about, when looking through the telescope, and observing whether the image appears to move relative to cross hairs.

The parallax is a source of error in observations. To eliminate the parallax, the telescope is pointed towards the sky and the eye-piece is moved in and out with a screwing motion until the cross hairs are seen in sharp focus. The telescope is then focussed upon some defined distant object so that the image formed appears clear and sharp. If the adjustment is correct, no relative movement between the image and cross hairs will exist by moving the eye about looking through the telescops.

MEASUREMENT OF HORIZONTAL ANGLES

Depending upon the accuracy and nature of work there are three methods of measuring horizontal angles in the horizontal plane, namely –

1. Ordinary method.
2. Repetition Method.
3. Reiteration Method.

1. Ordinary method : In this method the mean of the two readings with different faces

(face left and face right observations) gives the value of the required angle free from all instrumental errors.

A specimen *table for recording the observations* in this method is given below :

Instrument station	Straight to	Face Left			Face Right			Mean ∠AOB
		Vernier A	Vernier B	Mean Angle	Vernier A	Vernier B	Mean Angle	
0	A	00°00'00"	180°00'00"		00°00'00"	180°00'00"		35°37'20"
	B	35°37'20"	215°37'40"	35°37'30"	35°37'00"	215°37'20"	35°37'10"	

2. Repetition Method : In this method the same angle is mechanically added several times by keeping the vernier to remain clamped each time at the end of each measurement instead of setting it back to zero when sighting at the previous station. The correct horizontal angle is then obtained by dividing the final reading by the number of repetitions. Usually six readings, three with face left and three with face right, are taken. The average horizontal angle is then calculated.

The following tables give the method of recording *observations by the Repetition Method :*

TABLE - 1

Serial No	Instrument Station	Sighted to	Face Left readings				
			Vernier A	Vernier B	Total angle	No. of repetition	Mean horizontal angle
1.	0	A	0° 0' 0"	180° 0' 0"			
		B	35°30'20"	215°30'40"			
		A	35°30'20"	215°30'40"			
		B	71° 0'40"	251° 0'20"			
		A	71° 0'40"	251° 0'20"			
		B	106°31'40"	286°31'40"	106°31'40"	3	35°30'20"

TABLE - 2

Serial No.	Instrument Station	Sighted to	Face Right readings					Average horizontal angle
			Vernier A	Vernier B	Total angle	No. of repetition	Mean horizontal angle	
1.	0	A	0° 0' 0"	180° 0' 00"				
		B	35°30' 0"	215°30'20"				
		A	35°30' 0"	215°30'20"				
		B	71° 0'20"	251° 0'40"				
		A	71° 0'20"	251° 0'40"				
		B	106°30'40"	286°30'40"	106°30'40"	3	35°30'13"	35°30'17"

∴ Average horizontal angle = 35°30'17"

3. Reiteration Method : The method also known as *direction method* or *method of series* is suitable for the measurement of the angles of a group having a common vertex point. Several angles are measured successively in this method and finally the horizon is closed. The final reading of the leading vernier (vernier A) should be the same as its initial reading. If not, the discrepancy is equally distributed among all the measured angles.

The following tables are the *specimmens for reccording observations* by Reiteration Method :

TABLE - 1

Serial No.	Instrument station	Sighted to	Face Left readings				
			Vernier A	Vernier B	Mean angle	Correction	Horizontal angle
1.	0	A	0° 0' 0"	180° 0' 0"			∠AOB
		B	35°40'20"	215°40'40"	35°40'30"	-	35°40'30"
							∠BOC
		C	85°50'40"	265°50'40"	85°50'40"	-	50°10'10"
							∠COD
		D	145°57'20"	325°57'40"	145°57'30"	-	60° 6'50"
							∠DOA
		A	360°00'00"	180° 0'20"	960° 0'10"	-	214° 2'40"

TABLE - 2

Serial No.	Instrument station	Sighted to	Face Right readings					Mean Horizontal angle [F.L.+F.R.÷2]
			Vernier A	Vernier B	Mean angle	Correction	Horizontal angle	
1.	0	A	0° 0' 0"	180° 0' 0"			∠AOB	
		B	35°40'20"	215°40'40"	35°40'30"	-	35°40'30"	35°40'30"
							∠BOC	
		C	85°30'20"	265°50'40"	85°50'30"	-	50°10' 0"	50°10'05"
							∠COD	
		D	145°57'20"	325°57'20"	145°57'20"	-	60° 6'50"	60° 6'50"
							∠DOA	
		A	360°00'00"	180° 0' 0"	360° 0' 0"	-	214° 2'40"	214° 2'40"

The error at the close of horizon is negligible. No correction is required.

MEASUREMENT OF VERTICAL ANGLES

Vertical angle is the angle which the inclined line of sight to an object makes with the horizontal. It may be an angle of elevation or angle of depression depending upon whether the object is above or below the horizontal plane passing through the trunnion axis of the instrument. To measure a vertical angle the instrument be levelled with reference to the altitude bubble.

To measure angle of elevation or depression the following procedure is followed –

1. The theodolite is set up at station point and accurately centred, and levelled with reference to the altitude level.
2. The vertical verniers C and D are set exactly to zero by using the vertical circle clamp and tangent screw while the altitude level should remain in centre of the run.
3. The vertical circle clamp is released and the telescope is rotated in vertical plane so as to bisect the object. The vertical circle clamp is tightened and the object is correctly bisected by the slow motion screw.
4. The verniers C and D are read. The mean of the two readings is the value of the required angle.
5. Similar observations are made with the another face. The average of the two values thus obtained, is the value of the required angle which is free from instrumental errors.

Instrument Station	Sighted to	Face Left			
		Vernier C	Vernier D	Mean	Vertical angle
0	A	-5°12'20"	5°12'00"	-5°12'10"	
	B	+2°25'40"	2°25'20"	+2°25'30"	7°37'40"

Instrument Station	Sighted to	Face Right				Average Vertical Angle (AOB)
		Vernier C	Vernier D	Mean	Vertical angle	
0	A	-5°12'40"	5°12'20"	-5°12'30"		
	B	+2°26'00"	2°25'40"	+2°25'50"	7°38'20"	7°38'00"

MAGNETIC BEARING WITH A THEODOLITE

Although a theodolite is primarily an instrument for measuring horizontal and vertical angles, it is possible to measure the magnetic bearing of a line using the attachment known as tubular compass. This feature is particularly used when it is necessary to carry out a magnetic correction of underground correlations. The tubular compass may be attached to the standards of the theodolite as occasion arises.

When the tubular compass is used, the bearings are not read from the needle, but from the

vernier. The followings are the two methods for magnetic observations with a theodolite fitted with a tubular compass -

METHOD 1 : To take magnetic bearing the vernier A is set to zero, the needle allowed to rotate freely and the theodolite turned in azimuth until the end of the needle is centralised between the lines of the diaphragm. The vertical axis is clamped and now the telescope is pointing towards the magnetic north with vernier A reading zero. The vernier plate is released and instrument turned in azimuth to bisect the signal denoting the end of the line whose magnetic bearing required. The operation should be repeated a number of times to give an average reading.

The only disadvantage of this method is that the vernier is to be reset to zero and the compass needle recentred afresh for a series of magnetic bearings for averaging.

METHOD 2 : The method, Which reduces the amount of manipulation of the instrument and is more convenient and expeditious, is as follows -

The vernier A being clamped at zero, the other end of the survey line, whose magnetic bearing is required, is sighted. The vertical axis is then tightened, the vernier clamp realised, and the instrument is turned in azimuth until the compass needle is very nearly centralised between the diaphragm lines or is lying approximately in the magnetic meridian. The vernier plate is clamped and the diaphragm lines brought into the exact central position with respect to the end of the needle.

The reading of the vernier does not, in this case, indicate the clockwise magnetic bearing of the line, but the anticlockwise bearing assuming the horizontal circle of the theodolite is graduated clockwise, which is usual. It is then only necessary to deduct the reading 360° to obtain the magnetic bearing of the line.

The observations may be repeated by gently tapping the needle tube, causing it to swing slightly and come to rest in the magnetic meridian, centralising again by adjusting the vernier-plate tangent screw and reading the vernier.

If there is a change in the magnetic declination, the needle will move from its original position but it can be easily centralised again by adjusting the vernier-plate tangent-screw, when the new magnetic bearing may be observed from the vernier reading.

USING A THEODOLITE AS A LEVEL

If the theodolite is in perfect adjustment, it can be used as a levelling instrument. The theodolite is levelled with respect to the attitude level. When the bubble of the altitude level is in the centre and the vernier reading of vertical circle is zero, the line of sight is horizontal. Then the instrument can be used as a level.

The backsight (B.S.) is taken on the staff held at B.M. and the height of instrument is determined. The staff is then shifted to the point whose elevation is required, and a foresight (F.S.) is taken. Thus,

R.L. = B.M. + B.S. – F.S.

However, a theodolite should not be used as a levelling instrument as far as possible. The accuracy attained is generally less than that with a good dumpy level or tilting level.

TRAVERSING WITH THEODOLITE

TRAVERSE SURVEY : A *traverse survey* is one in which the framework consists of a series of connected lines, the lenghts and directions of which are measured with the help of tape or chain and an angle measuring instrument.

When the lines from a circuit which ends at the starting point, the survey is termed a **Closed Traverse**, if the circuit does not close, the traverse is known as **Open Traverse.**

The closed traverse is suitable for wide areas for locating the boundaries of mineral properties, lakes, woods, etc. The open traverse is carried out in the survey of long strips of country as in the case of canel, road, roadway, underground roadway, etc.

In theodolite traversing, theodolite is used for measurement of angles and tape or chain preferably steel tape is used for linear measurements. This method is applied for accurate and precise surveys.

Field Work : The field work in the theodolite traversing consists of -

1. Reconnaissance.
2. Selection, marking and referencing of stations.
3. Running of survey lines.
4. Picking up of the detail.
5. Booking of the field notes.
6. Office Work.

TRAVERSING WITH THE THEODOLITE

There are two systems of traversing by theodolite –

1. The continuous Azimuth Method, and
2. The Double Foresight Method.

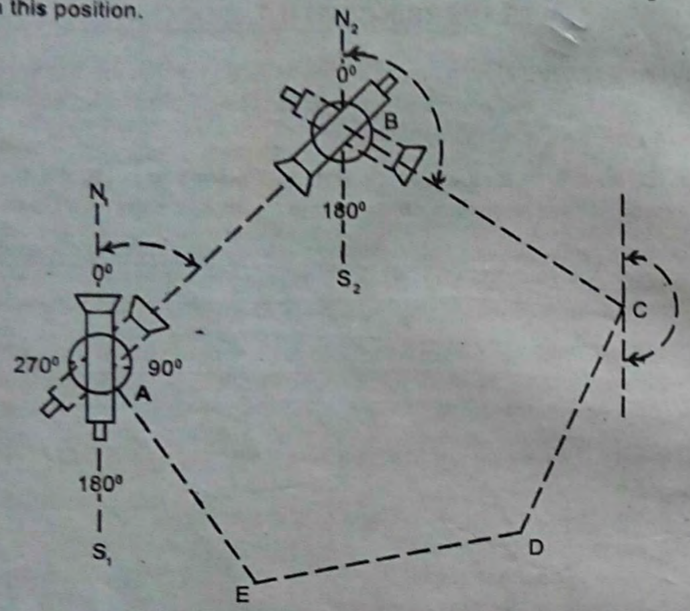
1. The continuous Azimuth Method : Let it is required to make a closed theodolite traverse of the lines joining the station points A, B, C, D, E.

The instrument is set up at station A and usual temporary adjustments are made for centring, levelling and removing parallax. The vernier is brought to read zero and it is clamped. the body clamp is realeased and the instrument rotated until the telescope is pointing north as shown by the attached compass with the vernier still reading zero.

The body is clamped and the vernier clam is released. The instrument is turned in a clockwise direction to sight the fore station B. The vernier is clamped and the station point is accurately bisected by the vernier tangent. The instrument now records the bearing of line AB.

If the bearing of the line AB is already known beforehand, the vernier reading is to be set accordingly before taking any sight from station B.

The instrument is next set up at B. The reading on the vernier is the fore bearing of AB. Body clamp is released and station is bisected using the body clamp and tangent. Body clamp is tightened in this position.



The telescope is now transited, vernier clamp released and station C is sighted. The vernier clamp is tightened and final bisection of the station point being obtained by means of vernier tangent-screw. The instrument now records the bearing of BC directly.

The instrument is then transferred to station C, and the telescope having been transited, a backsight is taken to B with the body clamp loose, and the vernier reading the bearing of BC. The body clamp is now tightened and final bisection of station B is done by body tangent-screw. The telescope is transited again and the vernier clamp is released to take a fore sight to station D. The vernier reading at once gives the azimuth of the line CD.

Thus, when similar operations are repeated at each station the forward azimuths of each draft are automatically recorded by the instrument.

To check accuracy of work after the traverse, the instrument should be set up at station A and the bearing of the first draft AB should be taken whether it gives the correct known bearing.

Advantages : (a) Bearings are obtained directly.

(b) Checking for magnetic bearing is possible at any iron-free station as work proceeds.

(c) Field work is expeditious.

(d) Calculations are simplified.

(e) Booking is simple.

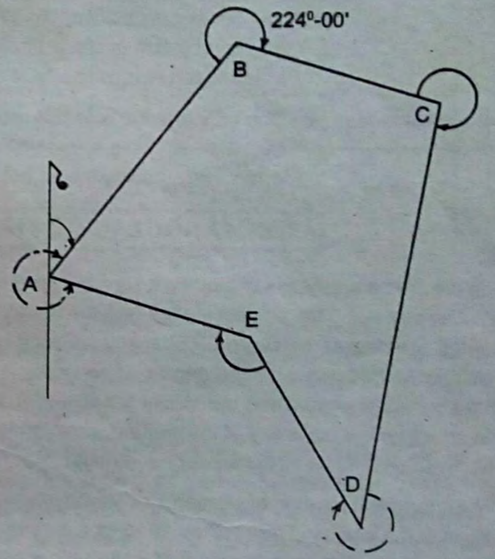
Disadvantages : (a) Double transiting at each station gives rise to error if the collimation adjustment is not perfect, the amount of error being twice the error in collimation.

(b) Any accidental shifting of the instrument at any stage affects the readings recorded.

(c) If a closing angular error is discovered, there is no sure way to ascertain whether it is cumulative, or occurred at one set up; nor, if the latter at which one.

2. The Double Foresight Method : In this method the bearing of initial line and the horizontal clockwise angles between adjacent lines are measured. The azimuths of all other lines are deduced by calculation.

Let it is required to make a closed traverse of the lines joining stations A, B, C, D, E by this method.



The instrument is set up at A and usual temporary adjustments are made. The A-vernier is set at zero and the instrument is directed towards the meridian as shown by the compass provided with body clamp and tangent. The body is clamped and the vernier is released. The telescope is now directed towards station B and final bisection of the station is made by the vernier tangent screw. The bearing of the line AB is recorded from the vernier. Alternatively, it may be assumed that the azimuth of the line AB is already known from previous survey.

The instrument is set up at B. The temporary adjustments having been made, the telescope is sighted to station A. Both plates being clamped, the two verniers are read and readings recorded. The usual practice is to set the A-vernier to zero, though this is not essential. Let the readings are :

A - vernier	B - vernier
00°0'00"	180°00'20"

The vernier plate is unclamped and the station C is sighted accurately by turning the telescope horizontally. The new vernier readings are recorded :

A - vernier	B - vernier
223°59'40"	44°00'40"

The difference between the readings in A vernier is 223°59'40" and that between B-vernier readings is 224°00'20". The mean of these difference is 224°00'00" and this is the correct value of the exterior clockwise angle between the lines BA and BC.

The instrument is shifted to station C and the operations are repeated so as to find clockwise angles BCD. The procedure is repeated at each station and the instrument being finally set up at A and the angle EAB measured.

If, the traverse had been made in the anticlockwise direction, the clockwise angles would still have been measured, but in such case the angles measured would have been the internal angles.

Advantages : (a) Any error in the line of collimation does not seriously affect the result.

(b) Any number of observations of each angle by the method of repetitions or by *'face left'* or *'face right' may* be made finally calculating the mean of the results to any required degree of accuracy is achieved.

Disadvantages : (a) The only drawback to this method for regular and extensive work is that it adds to the burden of office computation, which however, is often warranted by the higher degree of accuracy attainable.

RECTANGULAR CO-ORDINATES

In the ordinary process of plotting a survey by protractor and scale, each line of the survey is plotted by setting off the angle it makes with the meridian line and measuring its plan length to scale in the direction of the survey. Plotting by protractor is thus simple and convenient process, but it is evident that only a limited degree of accuracy is obtainable, depending on the scale to which the survey is plotted and the skill of the surveyor. Moreover unless the survey is a closed one, no check on the accuracy of the plotting is available, and if the bearing of the closing line is desired, this can only be obtained approximately.

An alternative method of plotting a survey involves the use of **rectangular co-ordinates** of each point in the survey. The method enables a survey to be plotted with great precision, each point may be plotted quite independently of any other point in the survey, the bearing and length of the closing line may be calculated to any required degree of accuracy consistent with the accuracy of the instrumental work. The area of any closed traverse may also be calculated from the known co-ordinates of the angular points. The use of rectangular co-ordinates must therefore be resorted to all important surveys.

RECTANGULAR CO-ORDINATES

Consider fig. 1 in which the line OA has a bearing of NαE and is L_1 metres long. Now it is clear that the point A is at a distance $L_1 \cos\alpha$ to the **north of O**, and also at a distance $L_1 \sin\alpha$ to the **east of O**. If these two distances be known, evidently a simple matter to fix the point A, and hence the line OA, quite independently of the angle α or of the length L_1 metres. It is only necessary

to measure the two distances $L_1 \cos\alpha$ and $L_1 \sin\alpha$ along the two axes NS and WE from the **Point of origin O** to fix the position of A.

The name rectangular co-ordinates is given to the two distances $L_1 \cos\alpha$ and $L_1 \sin\alpha$, $L_1 \cos\alpha$ is called the **latitude of A** with respect to O.

$L_1 \sin\alpha$ is called the **departure of A** with respect to O.

The latitude is measured to north (or south) of the east-west axis, and is a **northing or a southing** according to the quadrant in which the line under consideration occurs. North latitudes are regarded as positive and south latitudes as negative.

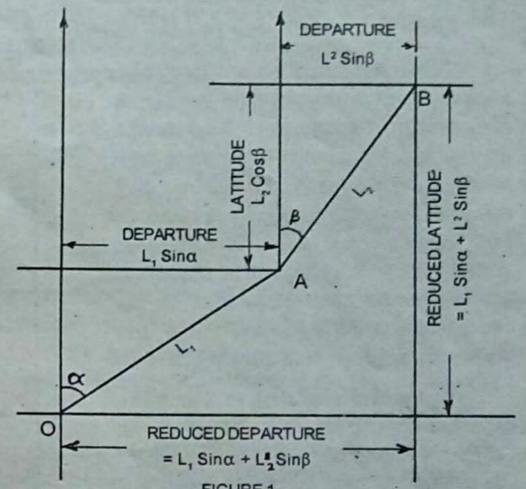


FIGURE 1

Similarly, **the departure** is measured to the east (or west) of the north-south axis, and is an **easting or a westing** as the case may be. East departures are regarded as positive and west departures as negative.

Let us now consider the second line AB the bearing of which is N β E and the length L_2 metres. **With respect to point A,** the latitude of B is obviously $L_2 \cos\beta$ and its departure is $L_2 \sin\beta$ and if these co-ordinates of B with respect to A are known the point B can be plotted by reference to axes drawn through A.

In practice, it is usually preferable to fix each point, not by reference to the point in the survey immediately preceding (i.e. not by reference to its **partial co-ordinates,** but by reference to its **total co-ordinates** or its co-ordinates based on the initial station, point of origin O). In other words, its **total latitude** and **total departure** from O must be known, and not merely the difference of latitude and difference of departure between A and B. Obviously, in the case given :

Total or reduced latitude of B = latitude of A + latitude of B.
$= L_1 \cos\alpha + L_2 \cos\beta$

Total, or reduced departure of B = departure of A + Departure of B.
$= L_1 \sin\alpha + L_2 \sin\beta$

Similarly, the total co-ordinates (or independent co-ordinates) of any point in a survey are equal to the sum of the partial latitudes and departures also known as consecutive or dependent co-ordinates of all the points preceding it. In making this addition due regard must be paid to the sign of the various latitudes and departures, whether north (positive), East (positive), or South (negative) and West (negative). In other words, it is **the algebraical sum** that must be considered, not the arithmetical sum.

The general rules to obtain the partial latitude and partial departure of each point of a survey are –

Latitude = plan distance x cos. bearing

Departure = plan distance x sin. bearing

These two simple equation may be applied successively to all the bearings and distance in a survey in order to calculate the partial co-ordinates of each point. The bearings referred to are quadrant bearings and if the bearings are expressed in azimuth, they must be converted to quadrant bearings before the co-ordinates can be calculated in this way. The quadrant in which the bearing occurs determines whether the calculated latitude is North or South, and the calculated departure East or West.

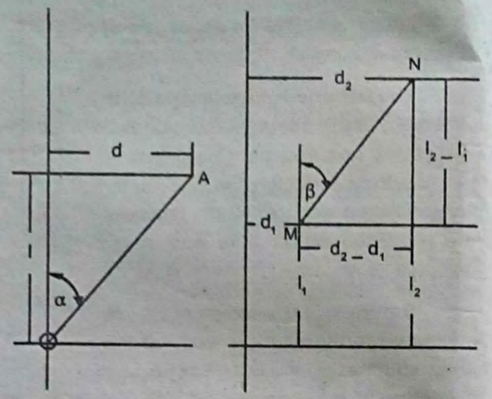


Figure 2

If the latitude of A is *l* and the departure is *d (Fig. 2)*, then, by Pythagoras' theorem the length of OA = $\sqrt{l^2 + d^2}$

Again, $\tan\alpha = \frac{\text{departure of A}}{\text{latitude of A}}$

or, the bearing of OA = $\tan^{-1}\frac{d}{l}$

Similarly, if the latitude and departure of M are l_1 and d_1, and the reduced co-ordinates of N are l_2 and d_2; then the length MN = $\sqrt{NY^2 + MY^2}$

$= \sqrt{(l_2 - l_1)^2 + (d_2 - d_1)^2}$

$= \sqrt{(\text{difference of latitudes})^2 + (\text{difference of departure})^2}$

And, the bearing of MN = $\tan^{-1}\frac{MY}{NY} = \tan^{-1}\frac{d_2 - d_2}{l_2 - l_2}$

$= \tan^{-1}\frac{\text{difference of departures}}{\text{difference of latitudes}}$

The following general rules may thus be expressed :

Tan angle bearing = $\frac{\text{departure}}{\text{latitude}}$

Plan distance = $\sqrt{\text{latitude}^2 + \text{departure}^2}$

[or plan distance = latitude x *secant* of bearing]

CLOSING ERROR

If a closed traverse is plotted according to the field measurements the end point of the traverse will not coincide exactly with the starting point, owing to the errors in the field

measurements of angle and distances. Such error is known as **closing error.**

In a closed traverse, the algebraic sum of the latitudes (i.e. ΣD) should be zero and the algebraic sum of the departures (i.e. ΣL) should be zero. The error of closure for such a traverse may be ascertained by finding ΣL and ΣD, both of these being the components of error 'e' parallel and perpendicular to the meridian.

Closing error = e = $\sqrt{(\Sigma L)^2 + (\Sigma D)^2}$, and

The direction of closing error = $\tan^{-1}\frac{\Sigma D}{\Sigma L}$

The sign of ΣD and ΣL will define the quadrant in which the closing error lies.

Adjustment of Angular Error : Before calculating latitudes and departures, the traverse angle should be adjusted to satisfy geometric conditions. In a closed traverse, the sum of interior angles should be equal to (2N - 4) right angles (or the algebraic sum of the deflection angles should be 360°). If the angles are measured with the same degree of precision, the error in the sum of angles may be distributed equally to each angle of the traverse. If the angular error is small, it maybe arbitrarily distributed among two or three angles.

Adjustment of bearings : In a closed traverse in which bearings are observed, the closing error in bearing may be determined by comparing the two bearings of the last line as observed at the first and last stations of traverse. Let 'e' be the closing error in bearing the line of a closed traverse having N sides.

Correction for first line = $\frac{e}{n}$

for second line = $\frac{2e}{N}$

for third line = $\frac{3e}{N}$

for last line = $\frac{Ne}{N} = e$

BALANCING THE TRAVERSE

'Balancing' is the term applied to operation of applying corrections to co-ordinates so that ΣL = ΣD when the survey form a closed polygon. The following are the common methods of balancing or adjusting a traverse.

1. Bowditch's method.
2. Transit Method.
3. Graphical Method.
4. Axis Method.

1. **Bowditch's Method :** The Bowditch's rule is commonly used to balance a traverse where linear and angular measurements are of equal precision. It distributes the total error in proportion to the length of drafts.

$$\text{Cor.} = \frac{e}{L} \times S$$

Where, Cor. = Correction to a Lat (or Dep).

e = Total error in sum of Lats (or Deps) with sign changed.

L = Total length of survey.

S = Length of particular draft.

This method is considered more mathematically correct but it changes the latitudes and departures in such a way that both the bearings and lengths of the courses are changed.

2. **Transit Method :** This method may be applied where angular measurements are more precise than the linear measurements. According to this method, the total error in latitudes and departures is distributed in proportion to the latitudes and departures of the sides. The latitudes and departures are adjusted in such a way that the length of the drafts are changed slightly but the bearings remain almost the same.

$$\text{Cor.} = \frac{e}{L} \times S$$

Where, Cor = Correction to a lat (or Dep).

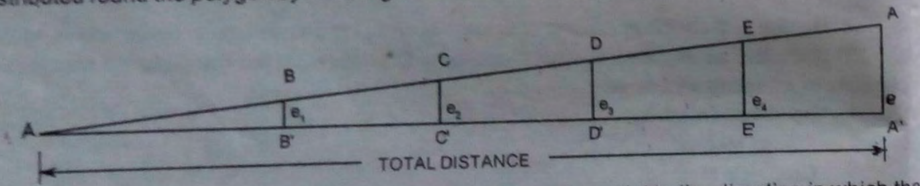
e = Total error in sum of Lats (or Deps) with sign changed.

L = Sum of Lats (or Deps) without regard to sign.

S = Length of particular Lat (or Dep).

3. **Graphical Method :** For rough survey, such as compass traverse, the Bowditch's rule may be applied graphically without calculating Latitudes and departures, etc. All the angles and bearings should, however, be adjusted to satisfy geometric conditions of the traverse before plotting the traverse for balancing by this method.

The traverse is plotted as AB'C'D'E'A' and the closing error A'A =e is found out which is distributed round the polygon by effecting a shift at each station proportional to the total distance



of that station from the commencement of the traverse and parallel to the direction in which the final shift is made. The correction at all the stations must be parallel to A'A.

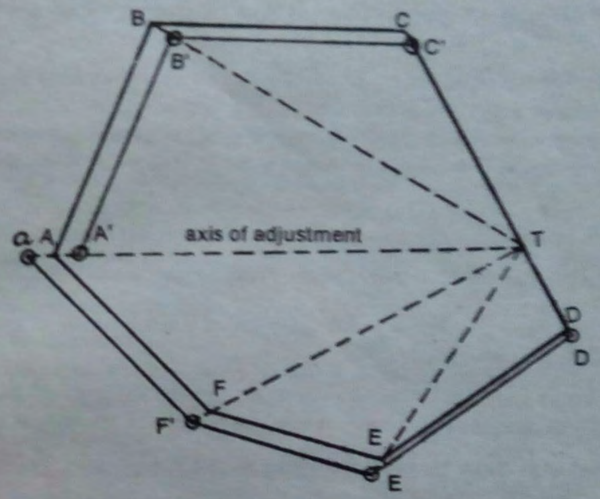
4. **Axis Method :** This method is adopted when the angles are measured very accurately. The directions of the line and the general shape of the diagram are not changed as the corrections are applied to the lengths only.

The closing error at A'-is joined and it is produced to cut the side C'D' at T. The line A'T is

the axis of adjustment, by which the traverse is approximately divided into two equal parts.

aA' is bisected in A, and TB', TE' and TF are joined.

Through A, a line AB parallel to A'B' is drawn to cut TB' produced in B. Through B, a line BC parallel to B'C' is drawn to cut TC' and produced in C.



Similarly, through A, the line AF parallel to aF' is drawn to cut TF' in F.

In this way the entire traverse is adjusted making the final traverse as ABCDEF.

In some cases the closing error oA' may not cut the traverse or may cut it in very unequal parts. In such cases the closing error should be transferred to some other convenient point.

SOURCES OF ERROR IN THEODOLITE SURVEY

The main sources of errors in traversing with a theodolite may be divided into -

1. Errors due to linear measurement, and
2. Errors due angular observation.

The sources of errors and the measures to be taken to eliminate them may be summarised as follows :

1. Linear Measurements : Different sources of errors and precautions to be taken are -

(a) Errors in alignment : If the tape is not accurately aligned between the stations being measured, the actual measured distance will be too long. If 'd' is the displacement error at end of tape of length 'L', the resulting error in length = $\frac{d^2}{2L}$

(b) Error due to faulty standardisation : This error will be constant for a given tape. The

error can be determined by comparison with a standard tape which is kept for standardisation only.

(c) Error due to variation from standard temperature : Unless the temperature is observed during measuring operations, constant errors will arise. Errors in measuring temperature of tape may vary for each tape length and if its probable value is $\pm t_2$ the error in measurement per tape length L will be $\pm L\alpha t_2$ where α = co-efficient of thermal expansion of the material used in tape. For steel $\alpha = 0.00012$ per $1^{\circ}C$.

(d) Errors from slope measurement : There may be a constant error due to slope of same magnitude and same sign if the whole length of line has the same inclination. If the whole of length has constant inclination and the heights of targets sighted are approximately equal, error in measurement of slope is given by $L(1-\cos\theta)$. Where L = length of slope and θ = grade of the section.

(e) Errors by variation from standard tension : There may be an error if the pull applied to the tape is different from the standard. The error may be expressed as $\pm\frac{(P-P_0)L}{AE}$, where P = pull applied during measurement, P_0 = Standard pull, A = cross-sectional area of tape and E = modulus of elasticity of the tape.

(f) Errors from sag : Error in pull will effect sag correction. Error may be constant for each spring balance and second error will vary in sign and magnitude from bay to bay.

(B) Errors in Augular Measurement may be due to –

(a) Instrumental error : The effect of residual errors of adjustment as well as non-adjustable errors can be reduced by averaging the face left and face right observations and reading opposite verniers.

(b) Errors of manipulation : There may be due to (i) Defective centring : If the centre of the instrument is not vertically over the station point, the angle of bearing is not measured from the point to which it is presumed that the foresight has previously been taken. The angular error depends upon error in centring, length of sights, the magnitude of angle being measured and position of instrument relatively to station and points sighted. (ii) Defective levelling : Due to incorrect levelling of the instrument, axis of rotation shall be inclined to vertical and horizontal axis of rotation shall be inclined. Effect of this error is more pronounced when angles are being measured between points of considerable different elevations. (iii) Slip : Observations are vitiated if the instrument is disturbed by turning (a) on tripod head by not being screwed firmly home so that it slackens back, (b) On lower parallel plate of a four screw head by levelling screws not being turned until they grip, (c) by neglecting to clamp shifting head, (d) by insufficient tightening of a lower clamp, (e) using wrong tangent screw.

(c) Observational errors : These consist of errors and mistakes in sighting and reading and include (i) inaccurate bisection of signal, (ii) non-verticality of signals, (iii) errors in reading of verniers, (iv) errors due to displacement of pegs or signals. For this purpose the instrument stations are chosen on firm ground, (v) Errors due to wrong bookings which can be avoided by

adopting a systematic manner of booking and repeating observations.

(d) Errors due to natural causes : Effects of high winds, refraction etc. can be minimised by avoiding field work during high winds and during midday especially during summer season.

QUESTIONS & ANSWERS

Question 1. Discribe various uses of theodolite in mine surveying. Explain why a theodolite is more accurate than other instruments.

Answer : USES OF THEODOLITE IN MINE SURVEYING

The theodolite is the most important instrument used for measurement of horizontal and vertical angles accurately for most purposes in mine surveying.

It is used for-

1. Triangulation.
2. Surface and underground traverses.
3. Correlation and joint surveys.
4. Trigonometrical levelling, cross-section levelling, contouring and for establishing grades.
5. Determination of magnetic bearing.
6. Tacheometric observations.
7. Locations of objects and measuring distances.
8. Calculation of areas.
9. Determination of the height of a chimney, and its deviation from the vertical.
10. Determination of true north, latitude and longitude at any place.
11. Curve ranging.
12. Laying out works and to locate position of shaft, headgear and winding engine.
13. Determination of fleet angle.
14. Driving of drifts and galleries in correct alignment.
15. Prolonging survey lines and locating points on a line.

Reasons for Accuracy of Theodolite : A theodolite is more accurate than other instruments due to -

1. Large diameter of plate circle reading from 10 cms to 20 cms.
2. Finely graduated circles to enable to read upto 20 minutes on the lower plate upto 20 seconds with verniers; 10 seconds or even 5 seconds can also be read with some verniers.
3. Hinged eye-piece arrangement for magnifying the reading or well-illuminated optical devices as in microptic theodolites.
4. High resolving power of the telescope :distant objects are very clearly focussed and accurately bisecterd.

5. Provisions of Ramsden eye-piece which gives image free from spherical and chromatic aberration.
6. Highly sensitive bubble tubes which enable the vertical axis of the theodolite to be made truely vertical.
7. Provision for altitude spirit bubble to eliminate error due to inaccuate levelling.
8. Provision of two verniers (A and B) to eliminate errors due to eccentricity of centres and eccentricity of verniers.
9. Provision of 'face left' and 'face right' observations by transiting the telescope to eliminate the errors due to imperfect adjustment of the instrument.
10. Fine centring arrangement including the optical plummet device which is very convenient in stormy weather.
11. Mechanical provision for fine adjustment.
12. Internal lighting arrangement in some instruments.

The modern theodolites have exceptionally sharp graduated circles made on glass and double reading arrangement. Telescope are made very bright by coating the lenses with reflection reducing materials, and its focusing is carried out by focusing rings on it. High quality apochromatic telescope ensures accurate bisection. By means of micrometer microscopes an angle can be read directly to the nearest second and 0.1 second by estimation.

Question 2. Describe with the aid of sketches, a theodolite.

Answer : DESCRIPTION OF A VERNIER THEODOLITE

The theodolite is one of the most important instruments used in mine surveying. It is used for measuring horizontal and vertical angle accurately. It can also be used for trigonometrical levelling. When it is equipped with stadia hairs, horizontal and vertical distances can be measured and when equipped with a compass, magnetic bearings can be determined.

An ordinary vernier theodolite used for more purposes essentially consists of three fundamental parts –

1. The levelling head at the base.
2. The horizontal circle or limb at the middle.
3. The alidate at the top.

The movements of these three parts are controlled by two clamps for rough adjustment, each equipped with a slow motion or tangent screw for fine adjustment.

The instrument is held on a tripod which may be either wooden or light metal alloy and the legs may be either rigid or telescopic. A small circular bubble is incorporated on the head of the tripod to enable the head to be approximately levelled.

1. **The Levelling Head :** It consists of -

(a) *Trivet stage* - A flat base of the instrument. It is threaded to screw the theodolite on the tripod.

(b) *The tribrach* - It is situated above the trivet stage. The tribrach and the trivet stage are connected through the levelling screws. A fine centring device is also incorporated in some theodolites whereby the instrument can be shifted exactly over or under the station point and clamped in position without disturbing the levelling.

2. **The Horizontal Circle or Lower Plate** - It fits into the central sleeve of tribrach and rotates within a collar controlled by the lower clamp and lower tangent screw. It is used for measuring horizontal angle and is graduated clockwise from 0° to 360° to read upto 20 minutes.

3. **The Alidade :** It consists of –

(a) *Vernier plate* - It is also known as upper horizontal plate or cover plate. It covers the lower plate and is carried on a spindle which rotates inside the hollow axis of the lower plate. The upper and lower horizontal plates can be clamped together by means of a clamp and the fine adjustments being done by the tangent screw. It carries two verniers each of which is fitted with a hinged microscope, two spirit levels and the standards to carry the telescope. Verniers are used to read upto 20 seconds. or in some cases upto 10 seconds.

(b) *The Plate Bubble tubes* - These are fitted on the upper surface of the cover plate with their longitudinal axis at right angles to each other for levelling the instrument.

(c) *Standards* - The vernier plate and the standards are cast in one piece. The telescope is supported in the standards by means of an axis called the trunnion axis fitted at right angles to the length of the telescope. These standards are sufficiently high to allow the telescope to transit.

(d) *Telescope* - This is usually an internal focusing type and consists of a large object glass (compound lens with an outer double concave lens and an inner convexo-concave lens), an internal focusing lens (double concave) moved by rack and pinion arrangement, a diaphragm and an an eye-piece of Ramsden type. The telescope can be rotated in a vertical plane on the horizontal axis.

(e) *Vertical Circle* - This is mounted on the telescope's horizontal axis and attached to the telescope so that when the telescope is elevated or depressed, the vertical circle moves with it. The two verniers of the circle, which can read upto 20 seconds, are mounted on a T-frame fixed to one of the standards. When the telescope is horizontal the zero of the vertical circle coincides with the zero of the vernier, it is usually divided into four quadrants from 0° to 90° in opposite directions from the two zeros placed at the ends of the horizontal diameter of the vertical circle. By means of the vertical circle clamp and tangent screw, the telescope and with it the vertical circle can be accurately set up at any desired position in a vertical plane.

(f) *Altitude Spirit Bubble* - This is a long sensitive spirit bubble tube attached to the top of one of the standards. It can be centred by means of the clip screws. In some instruments it is set on the top of the telescope. The altitude spirit bubble controls the indices for reading refers to the true horizon.

The theodolite is set over a station by means of a plumb-bob suspended from a hook fitted underneath the vertical axis of the instrument. In addition to an ordinary plumb-bob, a theodolite

may have an optical plummet- a small telescopic sight mounted in a verical hole through the spindle and adjusted to coincide with the azimuth axis of the alidade. It is viewed through a horizontal eye-piece at the side of the alidade. Since the device does not swing, it is unaffected by wind.

Question 3. Assume you have just purchased a new theodolite what tests would you apply to the instrument before actually using it in a survey ?

Answer : TESTS OF A NEW THEODOLITE

As provision is made for the elimination of the serious errors by adjustment, but the influence of some defects in the non-adjustable parts of a theodolite causes considerable errors during its use in surveying. The important requirements of a newly purchased theodolite should be as follows :

1. The whole instrument must be stable and in proper order.
2. The levelling screws should have tightness of fit.
3. The clamp should have well gripping power.
4. Slow-motion screws should easily travel.
5. The movement of horizontal and vertical axes should have no stiffness.
6. The telescope should have clear focusing arrangement and diaphragm hairs should be clearly visible.
7. All movements of the instrument must be circular.
8. The instrumental imperfections should be thoroughly checked and they include :

(a) **Eccentricity of Centres** : The graduated and vernier plates should be truely concentric or their axes should have the same geometrical axis of rotation. The difference between the two vernier readings should remain constant in all positions around the circle.

(b) **Eccentricity of Verniers** : The zeros of the two verniers should be at the ends of the same diametral line, and their readings should exactly differ by 180° on different parts around the graduated circle.

(c) **Errors of Graduation** : The divisions of the circles should be correctly and evenly spaced. This should be checked by observing a test angle on different parts of the plate.

Question 4. What are the qualities of a good surveying telescope ?

Answer : QUALITY OF TELESCOPE

A good surveying telescope should possess the following qualities -

1. It should be free from spherical aberration.
2. It should be free from the aberration of sphericity.
3. It should be free from chromatic aberration.
4. The definition should be good.

5. The illumination should be in suitable proportion to the other qualities of the telescope.
6. The aperture of the object glass should be as large as practicable.
7. The magnification should be at its maximum compatible with a sufficiency of illumination.
8. The field of view should embrance as large an area as possible.

Question 5. What are the defects of simple lenses ? How are these corrected ?

Answer : DEFECTS OF SIMPLE LENSES

The defects of simple lenses include *chromatic and spherical aberration, coma, astigmatism, curvature of field and distortion.*

Some of the above defects are not considered so important as the general aim is to obtain good image at the centre of the field of view and only the first two, chromatic and spherical aberration, affect the sharpness along the optical axis. The others are, however, of some importance to the surveyor in tacheometry and star observations where readings and coincidences are required over a field of view.

1. **Chromatic Aberration :** It is a defect of a simple lens whereby a blurred or coloured image is formed due to rays of white light passing through the lens are split up into its component colours and various colour waves are bent by various amounts thus converging to different foci. This dispersion of white light by lens is known as *Chromatic Aberration.*

The defect is corrected by a compound lens, termed "**achromatic lens**",- the outer lens is a convex lens of hard, light crown glass and the inner is a convexo-concave lens of dense, heavy flint glass. The combination of two kinds of glass used for compound lens reassembles the separated colours and brings as white light to a single focus.

2. **Spherical Aberration :** It is a defect of a simple lens arising from the surface of lens. Rays of light passing through a lens whose surface is perfectly spherical do not converge to a common point for which sharp image is not formed. The rays passing through the outer edges of the lens proceed to a different point from those passing through the outer edges of the lens proceed to a different point from those passing through the centre. The defect is corrected by using compound lenses whose errors counteract each other, the rays which pass through the thick part of one lens going through the thin part of the next, and vice-versa. The defect may also be corrected by placing a stop in the telescope so as to cut off the rays passing through the edges of the lens, utilising only those passing through the centre. But it has the disadvantage that the full area of the lens cannot be utilised.

In the eye-piece of a telescope, the use of two plano-convex lenses with their convex faces towards one another and within two-thirds of the focal length of either, has the effect of practically eliminating spherical aberration. Such lenses or combinations are said to be "aplanatic".

3. **Coma :** This is one type of oblique spherical abberration and therefore concerns rays not parallel to the principal axis. The chief effect is towards the margin and can therefore be reduced by restricting the field of view. Normally, it is preferable to have the wider aperture and the better light than to dim the field, as most surveying readings are at the centre and rarely much away from it.

4. **Astigmatism** : It is an effect of spherical aberration of oblique rays requiring different positions for focussing different directions of lines on the object. It is not so important in surveying instruments and therefore the correction is not usually made in full. A lens combination eliminating the effect is called '*anastigmatic*' and has at least three components in one lens. The compound lens and the Ramsden eyepiece of the surveying telescope largely correct the defect and it may then be neglected near the centre of the field of view.

5. **Curvature** : It is another defect caused by spherical aberration of oblique rays whereby a plane object is reproduced on a spherical surface and therefore definition at the centre of the field of view is not obtained at the same time as at the margin. The Ramsden eye-piece is the general method of the practical reduction of this defect. The 'anastigmatic' lens does not necessarily correct curvature.

6. **Distortion** : This is another effect from spheical aberration of oblique rays. The straight lines of the margin of an object are reproduced as curves in the image towards the margins of the field whether the lenses are simple or achromatic. Distortion and most of the other effects can be reduced by restricting the aperture, but the consequent loss of light and the smallness of the defect at the centre of the field of view make such method unnecessary in the surveying telescope.

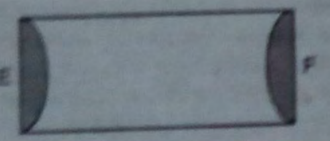
Question 6. Write short notes on :

(a) Ramsden Eye-piece.

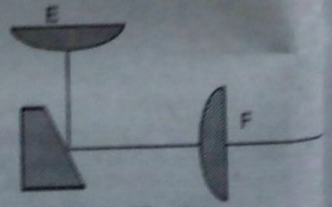
(b) diagonal Eye-piece.

Answer : EYE-PIECE OF A TELESCOPE

(a) **Ramsden Eye-piece** : In most surveying telescope, Ramsden eye-piece or so-called positive or non-erecting type eye-piece is used. It consists of two equal plano-convex lenses called the field-lens F, and the eye-lens E respectively placed with their curved surfaces facing each other. The distance between them being two-thirds of the focal length of either. This eye-piece gives a flat field of view and is very free from chromatic and spherical aberration. The inverted image formed by the object glass appears still inverted through this eye-piece which is the only disadvantage of this type of eye-piece. But for all precise work, the Ramsden eye-piece is to be preferred because for any desired magnifying power, the length of non erecting telescope is shorter and the definition is certain to be better.



Ramsden Eye-piece



Prism Diagonal Eye-piece

(b) **Diagonal Eye-piece** : When the line of sight is very much inclined to the horizontal, it becomes inconvenient for the eye to view through the ordinary eye-piece. In such a case diagonal eye-piece is used. The diagonal eye-piece generally of Ramsden type, consists of two

equal plano-convex lenses, placed with their curved surfaces facing each other but at right angle. A mirror or prism is used at an angle of 45° with the axis of the telescope. By this arrangement objects can be viewed in a direction perpendicular to the axis of the telescope. Such eye-piece is very much useful in astronomical observational and in metal mines for surveying highly inclined shafts, winzes or raises.

Question 7. Differentiate between an External-Focussing Telescope and an internal Focussing Telescope. What are the advantages and disadvantages of Internal-Focussing Telescope over External-Focussing Type ?

Answer : EXTERNAL-FOCUSSING AND INTERNAL FOCUSSING TELESCOPES

A surveying telescope may be either the external-focussing or the internal focussing type. The latter being the more modern but some what more complicated.

EXTERNAL-FOCUSSING TELESCOPE : In the external-focussing telescope, the barrel is formed of two tubes, one capable sliding axially within the other by means of rack gearing with a pinion attached to the milled focusing screw. The fixed outer tube is larger and carries the object glass at one end. The movable inner tube carries the smaller eye-piece at the opposite end and a diaphragm. If the telescope be directed towards a very distant point of light or object on its principal axis, the object entering the object-glass are practically parallel and are refracted or bent so that they all converge in a cone of rays to a point known as principal focus of the object glass and form there an image of the distant object. The diaphragm with its reticule of cross-wires is adjusted or brought to this point. After passing through the principal focus, the rays of light diverge and pass through the lenses of the eye-piece where they are refracted and present the eye with a magnified and an inverted image of the object sighted.

In some cases the objective cell is screwed to the inner tube, so that the focusing movement is effected by movement of the objective relatively to the outer fixed tube carrying the cross-wires and eye-piece.

INTERNAL-FOCUSSING TELESCOPE : This type of telescope is used on more modern instruments and differs from the external-focussing type in that the telescope consists of one tube of fixed length (apart from the eye-piece adjustment). Focussing is performed by a double-concave lens, fitted inside the telescope, which can be moved along the telescope by means of a milled-head screw. The introduction of a double-concave lens means that the rays of light are refracted to a second time but, after passing this lens, are less convergent. The rays of light, therefore, converage at the point 'principal focus' where the diaphragm is placed. The position of the double-concave lens can be altered, dependent on the distance of the object sighted, so that the image is always found at the diaphragm.

Advantages of Internal-Focussing Telescope : The advantages of the internal-focussing telescope are as follows -

1. The overall length of the tube is not altered during focussing, danger of breaking the plate level or upsetting the adjustment of instrument when transiting the telescope is thus eliminated.

2. The focussing slide is light in weight and located near the middle of the telescope.

Hence the balance of the telescope is not affected and the bubble is less liable to be displaced during focussing.

3. Wear on the rack and pinion is less due to lesser movement of negative lens.

4. Dirt, dust and water cannot enter the telescope as it is completely sealed.

5. Line of collimation is less likely to be affected by focussing. But on external focussing type the line of collimation may be altered if looseness between the tubes develop through wear.

6. In making measurements by the stadia method, an instrument constant is almost eliminated and the computations are thus simplified.

7. The negative focussing lens serves a useful optical purpose because the focal length of the combined objective and negative lens is greater than the distance between the objective lens and focal plane. This extra focal length can be utilised to increase the power.

8. Introduction of an extra lens helps to reduce the aberration of the system and to increase the diameter of the object-glass.

Disadvantages of Internal-Focussing Telescope : The principal disadvantages of an internal-focussing telescope are -

1. There is some loss of brilliancy in the image obtained due to the additional lens. This is compensated for by increasing the size of the object-glass.

2. The interior of the telescope is not so easily accessible for field cleaning and repairs.

3. With rapid changes in temperature and humidity, moisture condenses on the interior glass surfaces and dims the view.

Question 8. When would a theodolite fitted with (a) a side and (b) a top auxiliary telescope be used ? How would the observations be adjusted if each type of auxiliary telescope were used in observing both horizontal and vertical angles ?

Answer : THE AUXILIARY TELESCOPE

Although the diagonal eye-piece can be used for steep upward sighting it is of little use in mines where the roads are very steep and the theodolite has to be used for downward sighting. In such cases, especially in metalliferous mining the auxiliary telescope is often indispensable. When it is supplied with a special mining-pattern theodolite, it can be fitted either to one end of the transit axis or directly on to the main telescope. When it is attached to the transit axis it is known as a *side telescope* and transits independently in a plane parallel to that of the main telescope. If the auxiliary telescope is used in this position for measuring horizontal angles, no correction for eccentricity is necessary if the angles are measured for both faces and meaned.

When the auxiliary telescope attached to the main telescope, it is known as a *top telescope*. Its line of sight must be correlated with that of the main telescope and then it can be used for measuring horizontal angles without correction. If it is used for measuring vertical angles, however, a correction for eccentricity must be applied. This is a function of the distance between the centres of the main and top telescopes and of the distance from the instrument to the object

sighted. If, for example, the top telescope is offset 8 cms and an object 30 m away is sighted, the correction to the vertical angle will be the angle whose tangent is-

$$\frac{8}{30 \times 100} = 0.002666 = 00°09'10"$$

Since the main telescope can always be used for measuring angles of elevation, the top telescope would only be used for measuring angles of depression and in this case the correction would, therefore, be subtractive. All corrections can be avoided, however, if, when measuring angles involving steep sights, the auxiliary telescope is used in the top position for observing the horizontal and the side position for observing the vertical angles.

Question 9. Describe with the aid of sketches a type of optical device for centring fitted to a theodolite.

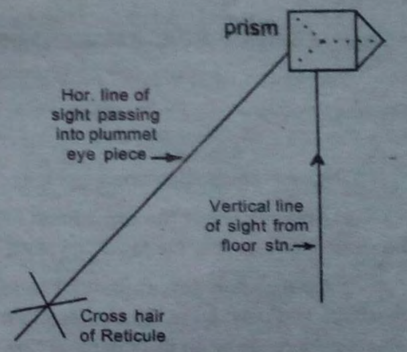
Answer : OPTICAL PLUMMET

An optical centring device, or simply an optical plummet, is a device for accurately centering a theodolite over or under a survey station. Centering by plumb-bob is difficult in windy weather or in mine road-ways where the velocity of air current is high. The development of optical plummet has resulted in far more accuracy being achieved in centering.

It consists of a horizontal telescope fitted with a right-angled prism in front of its object glass, and cross hairs with appropriate eye-piece focusing. If the plummet is levelled and in adjustment the line of sight of its telescope is horizontal and the rays of light from the station are then vertical. A theodolite having optical plummet built into its base for centering over a floor station, has a hollow vertical axis so that floor station can be viewed in the conveniently placed eye-piece on its upper plate. The image of the station is magnified by lenses and is viewed against the reticule carrying cross-hair, the intersection of which makes the instrument centre.

In some theodolites optical plummet is fitted to the main telescope to set instrument both over or under a station. The vertical axis of such instrument is hollow and a reversible spirit level is incorporated with the telescope to set it horizontal before centering and when the telescope is reversed the ground station can be observed.

Independent optical plummets are also available which can be used in conjunction with detachable levelling and centering head for up and down centering, especially used for surveying steeply inclined shafts and winzes in metal mines.



Optical Plummet

Extensive studies of various methods of centering have shown the following values of linear errors :

Plumb-bob centering accuracy - 1.2 to 1.5 mm.

Optical plummet accuracy - 0.8 to 1.0 mm.

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Question 10. Write short notes on :

(a) Trough Compass.

(b) Tubular Compass.

Answer : TROUGH COMPASS AND TUBULAR COMPASS

(a) **Trough Compass :** It is a rectangular wooden glass-covered box, 15 cm x 3 cm x 2 cm, having graduations on silver on either side of zero at the inside ends, and containing a powerful and sensitive magnetic needle, 12.5 cm long. The construction of the box is such that when the needle points zero, its axis is parallel to the sides of the box along which the north line may be drawn. The magnetic needle of the trough compass should play freely, be strongly magnetised, be very sensitive, and yet settle quickly.

(b) **Tubular Compass :** Modern theodolites are fitted with a compass of the tubular type and it is screwed to one of the standards. The tubular compass which is an improved type of trough compass consists of -

(i) A powerful and sensitive magnetic needle fitted inside a cylindrical brass tube, the north-end of which carries a pointer and which may be raised by a lifter controlled by a screw.

(ii) A circular glass diaphragm fitted in the end of the tube facing the pointer. On the diaphragm are etched two fine vertical parallel lines. The direction of the magnetic meridian is shown when the pointer is exactly midway between the two vertical lines; and

(iii) A small prism fitted inside the top of the tube to enable outside light to illuminate the diaphragm lines.

Question 11. Describe the permanent adjustments of theodolite and explain how each of the adjustments is effected.

Answer : PERMANENT ADJUSTMENT OF THEODOLITE

Permanent adjustments are those which are made by the maker, and only tested and corrected occasionally by the observer at comparatively rare intervals when he has reason to think that the instrument has been shaken by some means, and the adjustments are out.

The permanent adjustments of theodolite consists of the following –

1. **Adjustment of the plate level :** The axes of the plate levels must be perpendicular to the vertical axis so that the plate can revolve in a truly horizontal plane about the vertical axis.

2. **Collimation adjustment :** The line of sight must coincide with the optical axis of the telescope and must be at right angles to the horizontal axis of the telescope so that when the telescope is revolved on its horizontal axis the line of collimation generates a plane.

3. **Horizontal axis :** It must be perpendicular to the vertical axis and be truly horizontal.

4. **Telescope level :** The axis of the telescope level must be parallel to the line of sight.

5. **Vertical circle or index :** The reading of the verniers on the vertical circle must be zero when the line of sight is horizontal

TESTS AND ADJUSTMENTS

1. **Adjustment of the Plate Levels :** The instrument is set up on firm ground as accurately level as possible by the eye by manipulating the tripod legs. The lower clamp is tightened and the vernier clamp is loosened. The vernier plate turned until one of the plate levels is parallel to any pair of levelling screws. Each plate bubble is then brought to the centre of its run turning the levelling screws.

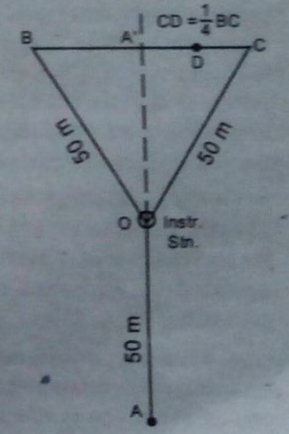
The instrument is now rotated about the vertical axis through 180°. If the bubbles remain central the adjustment is correct and the axes of plate levels are perpendicular to the vertical axis and the vertical axis is truly vertical.

But if the adjustment is not correct, the deviation of the bubble is noted. Each bubble is then brought halfway back by means of capstan screws at the end of the bubble tube and rest by means of respective levelling screws. The test and adjustment are repeated until both bubbles remain central during a whole revolution of the instrument.

2. **Collimation Adjustment :** The adjustment is made in the following steps :

(a) *Adjustment of horizontal cross-hair :* To make the horizontal cross-hair actually horizontal, any point, within the field of view and lying on the cross-hair is selected. The telescope is slowly moved in azimuth and the movement of the point in relation to the horizontal cross-hair is observed. In case, the point is found to move away from the cross-hair during rotation, the cross-hair is not horizontal. The four set-screws holding the ring and containing cross-hair are loosened and the ring is rotated slowly in the required direction, so as to eliminate the observed tilt, and the point under observation then moves on the horizontal cross-hair when the telescope is rotated horizontally.

(b) *Adjustment of vertical cross-hair :* The instrument is set up and levelled carefully. The vertical hair of the diaphragm cross-hairs is sighted on a well-defined point. Both the plates are clamped and the telescope is moved up and down. If the point does not appear to travel along the vertical cross-hair, the screws holding vertical cross-hair are loosened and by tapping lightly on one screw, the ring is rotated slowly until the point appears to travel along the vertical cross-hair. After completion of adjustment the screws are tightened without disturbing the diaphragm.



The instrument is finally set up at a convenient point on a fairly level ground after the adjustments for the axes of plate levels and also for the horizontal and vertical cross hairs. Keeping the telescope nearly horizontal, station mark A, at a distance of about 50 metres is sighted and brought into focus, without parallax. Both the body and vernier plates are clamped. The telescope is transited and made more or less horizontal. Any station B, at a same distance of about 50 m and at about the same level as A, is

fixed in the alignment of the cross-hairs. The vernier clamp is released and station A is sighted back by turning the telescope in azimuth through 180°. The telescope is again transited. If the instrument is in adjustment, the station B should again lie on the cross hair.

But if the adjustment is not correct, the station B should not lie on the cross-hair.

To make the necessary adjustment, a third station point C is fixed in the line of sight and at about the same distance (50 m) from the instrument. The distance between B and C is measured and at $\frac{1}{4}$ th this distance from C, a point D is located. The vertical hair is moved to coincide with D by loosening the diaphragm screw holding the cross-hair ring in one side and tightening the opposite one. The same procedure is repeated until the adjustment is perfect in which case the points A, O and A' will remain in same line.

3. **Adjustment of Horizontal Axis :**

Spire test : The theodolite is set up on a fairly level ground at about 10 m away from a high building or chimney and levelled it very carefully. A well-defined point 'A' at the top of the building or chimney is sighted. Both the clamps are tightened. The telescope is lowered and a point B in line of sight about level with the telescope is marked on the wall. The lower plate is loosened. The telescope is transited and turned about 180° horizontally. The point A is sighted back with the telescope and the lower clamp is tightened. The telescope is lowered to intersect the point B previously marked. If the point B is not coincident with the line of sight the instrument requires adjustment.

The adjustment is effected by raising or lowering the adjustable end of the horizontal axis by the capstan-headed screw near the top of the standard. The adjusting screws are brought in position by a right-hand turn, otherwise its block on which the horizontal axis rests might stick and not follow the screw. After the adjustment made, the capstan screws are tightened just enough to avoid looseness of the bearing.

4. **Adjustment of Telescope Level :** For the adjustment two stations A and B are pegged down on a fairly level ground, about 100 m apart. The instrument is set up in a mid position, between A and B, and temporary adjustments performed. The vertical circle is clamped and the telescope bubble is brought exactly to the centre of its run by means of tangent screw of the vertical circle. With the bubble exactly central, readings on the staff held on A and B are taken. Since A and B are equidistant from the instrument, the difference between these readings indicate the true difference of level between A and B.

The instrument is moved to a point P on the line BA produced, at about 20 m apart from A. The instrument is levelled accurately with the telescope bubble exactly central, the readings on the staff held first on A and then on B are noted. The difference between the two readings should agree with the first or true difference of level between these points to indicate the adjustment correct.

If the instrument is not correct, the correct staff readings on A and B are calculated. The horizontal cross-hair is brought exactly to the correct reading on B by means of tangent screw of vertical circle. The bubble is then brought centre of its run by means of the capstan-headed screw of the level tube.

The staff held on the near station A is sighted and the reading noted to get the calculated correct reading. The procedure is repeated until the test is satisfied.

5. **Adjustment of Vertical Circle or Index :** The instrument is set up and its plate bubbles are carefully centered. The telescope bubble is brought to the centre of its run by means of the vertical tangent screw. The vernier on vertical circle should now read 0°. If not, adjustment is required for this index error. For adjustment the screw holding the vernier are loosened and it is lightly tapped until the zeros coincide.

Question 12. Explain clearly what would be the effect of the following when measuring horizontal and vertical angles with theodotite :

(a) Error in levelling the base plate.

(b) Lack of perpendicularity between the vertical axis and transit axis.

(c) Index error on the vertical circle.

(d) Failure to eliminate parallax in the telescope.

(e) Inaccurate centring.

(f) Non-verticality of ranging rod.

Answer : EFFECTS OF ERRORS WHEN MEASURING ANGLES

(a) **Error in Levelling the Base Plate :** If the base plate is not horizontal when the bubbles in the plate level are centred, the vertical axis of the theodolite will not be truely vertical. The horizontal angles will be measured in an inclined plane and not in a horizontal plane. The vertical angles measured will also be incorrect. The error may be serious in observing the points the difference in elevation of which is considerable. The error cannot be eliminated by double sighting.

(b) **Lack of Perpendicularity between Vertical Axis and Transit Axis :** If the transit axis is not perpendicular to the vertical axis, the line of collimation will not revolve in a vertical plane as the telescope is raised or lowered. The resulting angular error affects both horizontal and vertical angles, and depend upon (i) the inclination of the transit axis to the perpendicular to the vertical axis, and (ii) the vertical angle to the point sighted. The error is considerable when points between which horizontal angles are to be measured are at very different elevations. No error, however, is introduced, if the two points between which horizontal angle is to be measured, are at the same level. The error is eliminated by making double face observations of the angle and taking the mean of the two values.

(c) **Index Error on the Vertical Circle :** If the vertical circle verniers do not read zero when the line of sight in horizontal, the vertical angles measured will be incorrect. The index error has no effect on horizontal circle reading.

(d) **Failure to eliminate Parallax :** Due to parallax, accurate bisection of the object is not possible. Hence the readings on the vertical circle and horizontal circle will be incorrect.

(e) **Inaccurate centring :** This error is introduced in all angles measured at a given station if the centre of the instrument is not exactly over the station. The magnitude of the error depends upon (i) the error of centring, and (ii) the length of the sight.

The error due to defective centring varies inversely as the length of the sights. The error will be more when the sight is short. When the error in centring is 1 cm and the length of sight is 36 m, the error in a sight is about 1' of arc. It is about 3' when the error of centring is about 1 cm and length of sight is 12 m.

(f) **Non-verticality of Ranging Rods :** The observed angles will be incorrect if the station mark is not bisected accurately due to non-verticality of ranging rod or due to some obstacles. The error varies inversely as the length of the line of sight. If the ranging rod put at the station mark is not held vertical, the error 'e' is given by

$$\tan e = \frac{\text{Error in verticality}}{\text{Length of sight}}$$

Question 13. Write short notes on :

(a) Residual errors.

(b) Index errors.

(c) Personal equation.

Answer : ERRORS IN ADJUSTMENT AND OBSERVATION

(a) **Residual Errors :** The adjustment of theodolite being very delicate, perfect adjustment is not possible in spite of utmost care taken in making the adjustment. Consequently, very small error remain unadjusted and these are called *residual errors.*

(b) **Index Errors :** It is the discrepancy between the zero of vertical circle and the arrow of the vernier when the theodolite and its telescope are truly horizontal.

(c) **Personal Equation :** different persons do not see things exactly alike. Each person may have an inaccuracy peculiar to himself. One may take an observation different from another equal in skill. Also one's right and left eye may differ. This difference in persons is called their *personal equation.*

Question 14. Describe the repetition and reiteration methods of measuring angles.

Answer : REPETITION AND REITERATION METHODS

REPETITION : When the same angle is measured more than once with different index readings on several parts of the graduated scale to eliminate error due to its imperfection by their average is known as repetition.

For very accurate measurement of angle the method of repetition is followed. Measurement of the angle clockwise is repeated several times in such a way to add up successive readings on the horizontal scale and the final reading is divided by the number of observations taken to get the mean. If desired, both the vernier readings may be noted, whilst the whole procedure may be repeated with reversed face.

Advantages : The following errors are eliminated or minimised by method of repetition -

1. Errors due to the eccentricity of centres and verniers are eliminated by taking both vernier reading

2. Errors due to inadjustments of line of collimation and the trunnion axis are eliminated by taking both face readings.
3. The error due to inaccurate graduations are eliminated by taking the readings at different parts of the circle.
4. Errors due to inaccurate bisection of the object, eccentric centring etc. may be to some extent counter-balanced in different observations.

Disadvantages : The disadvantages in repeating angles are -

1. Errors of observation are accumulated, as an observer tends to make the same error of observation each time the telescope is directed upon the same object.
2. Errors due to instability of clamping and tangent screws are also introduced.
3. Errors due to slip, displacement of station signals and want of verticality of the vertical axis, etc. are not eliminated since they are cumulative.

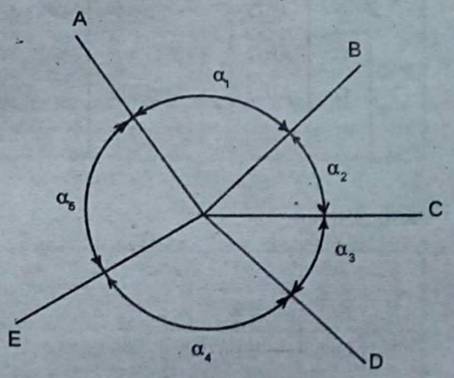
In theory, the method of repetition is apparently perfect, but from the defects of materials and workmanship and from want of stability of different parts of the instrument, it is found in practice to repeat the error along with the angle measured.

Uses : The repetition is used in correlation work, triangulation, measurement of same angles, laying out important projects, etc.

REITERATION : This consists of taking a number of independent observations of the angle to be measured, using preferably a different part of the graduated circle for each observation and making an equal number of face left and face right observations.

In figure, Angle α_5 measured for closing the horizon

Angle $\alpha_1 + \alpha_2 + \alpha_3 + \alpha_4 + \alpha_5 = 360^0$.



Several angles are successively measured clockwise at a station and finally the angle between the last station and the initial station is measured and the horizon is closed. The final reading of the vernier should be the same as its initial reading. If not, and if the discrepancy is small due to slip or flexure, the error is equally distributed among the several observed angles. But if the error is large, the readings should be discarded and new set of readings should be taken in lieu of previous observations.

The values of the different angles are deduced by subtraction and an average of the various vernier results for several reiterations may be taken as the correct. Usually two reiterations are sufficient for the purpose. Observations with both faces and readings both in clockwise and anti-clockwise directions may be made for precise work.

Advantages : The advantages of reiteration method are-

1. Error for slip due to defective clamping and tangent screws arrangements are eliminated.
2. Errors due to non adjustment of collimation, defective graduations and inaccurate centering and levelling are eliminated by taking face left and face right observations and by reading both verniers.
3. Personal errors in the bisection of and reading of verniers are also eliminated.
4. This method is much less tedious than repetition when a number of angles are to be measured at a station.
5. A ready check of the observations is possible.

Disadvantages : The disadvantage is that the degree of accuracy is requisite only in extensive operations.

Uses : This method of measuring angles is used when a large number of angles are to be measured at a station, in triangulation survey and in other precise works.

METHODS OF BOOKING :

Repetition –

Inst.Stn.	Sight Stn.	Angle	Vernier A	Vernier B	Mean Vernier	Difference	Face	Mean Angle
B	A		0°00'00"	180°00'00"			L	
	C	ABC	51°50'50"	241°50'00"	61°50'20"	61°50'20"	L	
	C	ABC	123°41'40"	303°-41'-40"	123°41'40"	61°51'20"	L	
	C	ABC	185°32'20"	5°32'40"	185°32'30"	61°50'50"	R	
	C	ABC	247°21'40"	67°-22'20"	247°22'20"	61°49'50"	R	61°50'35"

Reiteration –

Inst Stn.	Sight Stn.	Vernier A	Vernier B	Mean Vernier	Difference	Remarks
M	Left	0°00'00"	180°00'00"	0°00'00"	-	
N	"	91°31'00"	271°31'00"	91°31'00"	91°31'00"	
O	"	129°21'40"	309°21'40"	129°21'40"	37°50'40"	
P	"	284°48'40"	104°48'40"	284°48'40"	155°27'00"	
M	"	0°00'00"	180°00'00"	0°00'00"	75°11'20"	

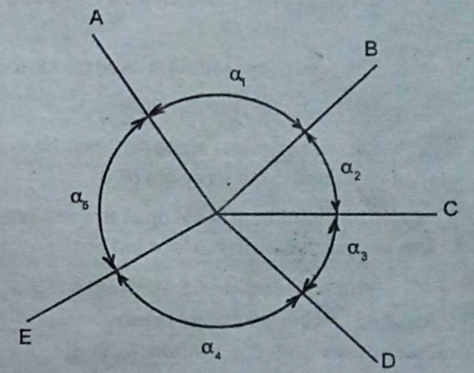
Question 15. State how you would proceed to measure four angles from a station by Reiteration method by using a Microptic Theodolite of 20" Least Count. Also give a sample chart of booking those observations by assuming your own figures. Show in the bookings the corrected angles after applying corrections in the errors of closure for each face and give the mean value.

Answer : MEASURING ANGLES BY REITERATION METHOD

Reiteration Method : The method consists of taking a round of angles clockwise at a station and measuring finally the angle between the last station and the initial station closing the horizon. The measurement of four angles from a station O is made as follows -

The instrument is set and levelled at O. The vernier plate is set to zero and it is clamped. The body clamp is released and the telescope is directed to A. Both the vernier readings are noted and the body (limb axis) is clamped.

The vernier clamp is released and the stations B, C, D, E, A, each in turn bisected, rotating the telescope in a clockwise direction. The readings of both verniers in each case are noted and finally the horizon is closed on the starting station A.



If the final reading does not agree with the first reading, the discrepancy is to be distributed equally among the several angles. Such discrepancy should be 20 seconds to 1 minute according to the class of theodolite used; otherwise the readings should be discarded and repeat observations made.

The values of the different angles are deducted by subtraction and an average of the various vernier results for several reiterations may be taken as the correct. Usually two reiterations are sufficient for the purpose. Observations with both faces and readings both in clock-wise and anti-clockwise directions may be made for precise work. The operations may be repeated but setting the vernier to some value other than zero degrees to eliminate errors of graduation.

BOOKING

Instr. Stn.	To Stn.	Face	Vernier A	Vernier B	Vernier Mean	Vernier Difference	Accepted Angle
O	A	L	0°00'00"	180°00'00"	0°00'000		
	B		91°31'00"	271°31'00"	91°31'00"	91°31'00"	
	C		129°21'40"	309°21'40"	129°21'40"	37°50'40"	
	D		208°18'40"	28°18'40"	208°18'40"	78°57'00"	
	E		272°30'00"	92°30'00"	272°30'00"	64°11'20"	
	A		0°00'00"	180°00'00"	0°00'00"	87°30'00"	

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The table shows the booking for one single round only. Other rounds on the same face an or opposite faces are made and booked likewise and the average of the values obtained in th last column (Accepted Angle) taken as the required values of the angles.

Question 16. Describe a method used for the measurement of a horizontal angle undergroun with a theodolite and make out booking of imaginary readings.

Answer : MEASURING HORIZONTAL ANGLES BELOWGROUND

There are two standard methods of measurement of angles with a theodolite. namely :

1. Repetition, in which a single angle is measured a number of times on different parts o the graduated circle.

2. Reiteration, in which a series of angles from the same station are recorded on the fu circle.

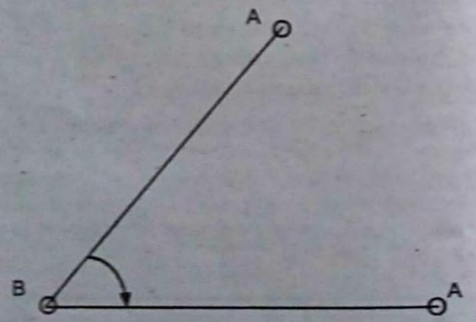
The repetition method is used for the measurement of a horizontal angle underground.

METHOD OF REPETITION

Let the horizontal angle ABC is to be measured, the instrument being at B, with A the bac sight (B.S) and C the foresight (F.S).

The instrument is set up at B, centred and levelled. The station A is sighted and bisecte with the instrument in the face left position and with the vernier at zero.

The vernier clamp is then released and the instrument is turned clockwise, and the station C is sighted and bisected. The horizontal circle on both verniers being read and recorded.



The body clamp is released, the telescope is transited and the B.S. sighted and bisected, with the instrument in the face right position. The horizontal angle then checked which should be the same as it was at the F.S. reading.

The vernier clamp is released and the instrument is turned clockwise, the F.S. being sighte bisected and the horizontal circle read on both verniers and recorded.

The above procedure gives one F.L. and one F.R. value for the angle and this procedure repeated as often as necessary to give mean angle consistent with the accuracy required for th work.

Booking

Inst. Stn.	Obs Stns.	Face	Horizontal Circle Rdg.	Reduced Hor. Angle	Mean Face Angle	Accepted Angle
B	A	L	00°01'20"			
	C		69°41'20"	69°40'00"		
	C	R	139°21'40"	66°40'00"	F.L. 60°40'10"	
	C	L	209°01'40"	69°40'20"		69°40'00"
	C	R	278°41'20"	69°39'40"	F.R. 69°39'50"	

Question 17. Describe a method to find out as accurately as possible the distance between two inaccessible points.

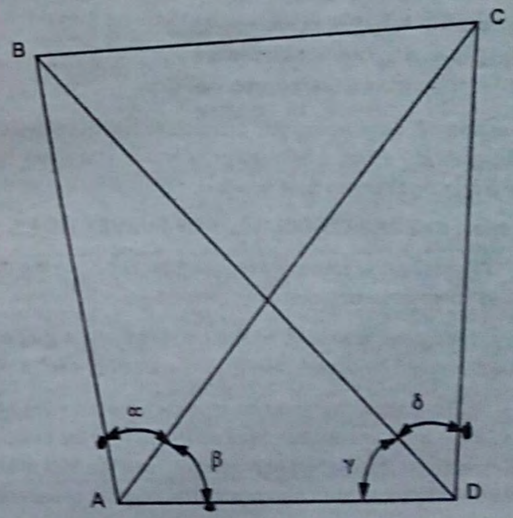
Answer : INACCESSIBLE DISTANCE

Sometimes it may be necessary to determine the distance between two points which are not only inaccessible to each other, but are isolated from the surveyor by natural features.

Let B and C are the two inaccessible points between whom the distance is to be found out.

A stretch of fair level ground is selected and on this two intervisible points A and D are fixed from which both points B and C are visible. AD now formed the base line and the distance between A and D is then very carefully measured with a steel tape. The measured length is corrected for standardisation, temperature, tension, slope, sag, etc.

A theodolite is set up at A and after making necessary temporary adjustments horizontal angles BAC and CAD are measured with care. Mean of face left and face right values of angles are taken for ensuring accuracy.



The instrument is next shifted to station D and after necessary temporary adjustments horizontal angles ADB and BDC are accurately measured in a similar way.

Let, Angles BAC = α
CAD = β
ADB = γ
BDC = δ

Now, in triangle ABD,

$ABD = 180^0 - (\alpha + \beta + \gamma)$

By sine rule, $AB = \frac{AD \sin \gamma}{\sin [180^\circ - (\alpha + \beta + \gamma)]} = x$

Again, in triangle ACD, $\angle ACD = 180^\circ - (\beta + \gamma + \delta)]$

$$\angle ADC = (\gamma + \delta)$$

$$\therefore AC = \frac{AD \sin (\gamma + \delta)}{\sin [180^\circ - (\beta + \gamma + \delta)]} = y$$

By cosine rule, in triangle BAC, $BC = \sqrt{AB^2 + AC^2 - 2ABAC.\text{Cos } BAC}$

$$= \sqrt{x^2 + y^2 - 2x.y.\text{Cos } \alpha}$$

In this way the distance BC is calculated.

Question 18. Write short notes on : Bowditch's Rule.

Answer : BOWDITCH'S RULE

The following are the main points of Bowditch's Rule as applied to the adjustment of closing errors in traverses :

1. It is assumed that probable errors in length and bearing of a leg of a traverse produce equal displacements in length and bearing of the end point.
2. Errors in length are proportional to the square root of the length.
3. Errors in bearing are inversely proportional to the square root of the length.
4. Correction to a latitude or (departure) is given by -

$$\text{Cor.} = \frac{e}{L} \times S$$

Where e = Total error in sum of Lats (or Deps) with sign changed.

L = Total length of survey.

S = Length of particular draft.

Question 19. How would you proceed to check accuracy of your work after a closed traverse underground ? What maximum error would you allow for angular and linear measurements ? Show with an example how would you distribute the errors ?

Answer : CHECKING ACCURACY OF SURVEY WORK

The following steps are required for checking the accuracy of field wolk after completing a closed traverse underground :

1. **To draw a sketch of the traverse -** As a guide to computation, a rough sketch of the traverse should be drawn showing the names of each stations.

2. **To check the angular measurements -** (a) After completing a closed traverse the instrument work should be checked in the field by setting up the instrument at the first station and measuring the clockwise angle between the first draft and last draft. The calculated bearing of first draft should then agree with the original observed value. Any difference represents the aggregate angular error accumulated in the circuit.

(b) A further check depends on fact that the some of the internal angles of a polygon having

n sides is equal to $(2n - 4) \times 90^\circ$, and the sum of external angles is equal to $(2n + 4) \times 90^\circ$. Some slight discrepancy or closing angular error is to be expected. If it exceeds the limit of permissible error, the traverse has to be repeated.

3. **To reduce the linear measurements -** Actual measurements of each draft should be corrected for a standardisation, slope, sag, and for depth and scale factors. A corrected horizontal length of each draft should be calculated.

4. **To compute the directions or bearing -** Starting with an assumed or known bearing, the bearing of all the drafts should be computed by applying the corrected angles successively.

5. **To compute the co-ordinates -** After angular value has been corrected and horizontal lengths of each draft determined, the co-ordinates of all the traverse station should be computed. The algebraic sum of latitudes and also of departures should be zero. In practice, however, the figures never exactly balance and some error in latitude and also in departure are observed. The total closing error is evidently the square root of the sum of the squares of these values.

6. **To compute the measure of accuracy -** The ratio of the total error to the total length of the survey should be computed. If the ratio exceeds the permissible limit, a blunder probably exists and the survey should be rejected.

PERMISSIBLE ERRORS

Underground traverse which has been closed polygonally-

(a) **Angular error (Total)** : $(20 + x)\sqrt{n}$ seconds, where n is the number of observing stations and x = 10 seconds or smallest reading in second of the vernier (least count).

(b) **Error of closure** : The ratio of closing error (as found out from the computation of Lats, and deps) and sum of horizontal length of draft should not exceed 1 : 1500

DISTRIBUTION OF CLOSING ERROR

(a) **Angular error adjustment :** If the angular error is within the limit of permissible error, it should be distributed equally between the angular measurements on the assumption that the probability of error is equal at all the stations.

Station	Internal Angle	Correction	Corrected Angle
A	64°53'30"	-30"	64°53'00"
B	206°35'15"	-30"	206°34'45"
C	64°21'15"	-30"	64°20'45"
D	107°33'45"	-30"	107°33'15"
E	96°38'45"	-30"	96°38'15"
	Sum = 540°02'30"	-2'30"	540°00'00" Check
Sum of angles = (2 x 5 - 4) 90 = 540°00'00"			

Total error = 2'30" (Assuming that the error is within limit)

Error per angle = 2'30" ÷ 5 = 30"

(b) **Adjustment of error in Co-ordinates** - This consists of a reasonable process that makes their respective sums equal zero.

If the ratio of the closing error to the total perimeter, e : p (ascertained after distribution of the angular error) falls within the permissible limits, the errors in latitude and departure should be distributed among the various drafts. There are various methods of doing this, one of which is known as Bowditch's Rule which the total error is distributed in proportion to the length of the drafts.

$$\text{Correction in latitude (or departure)} = \text{Total error in latitude (or departure)} \times \frac{\text{Length of draft}}{\text{Perimeter or traverse}}$$

If the eastings are 0.8 m in excess of the westings, the sum of the measured distance = 1520 m and length of one of the drafts = 380 m. Then the correction that must be applied to the departure of this draft = $0.8 \times \frac{380}{1520} = 0.2$ m. This must be deducted from the departure of the draft, if easterly, and added, if westerly. In other words, due account should be taken of the sign in each case.

Bowditch's rule is considered more mathematically correct, but it changes the latitudes and departures in such a way that both the bearings and lengths of the draft are changed.

Question 20. Discuss briefly the types of theodolites used in triangulation of high precision.

Answer : TYPES OF THEODOLITE FOR PRECISE WORKS

There are three types of theodolites used in highly precise works. These are-

1. The Repeating Instrument.
2. The Direction Instrument.
3. The Direct scale Reading instrument.

1. **The Repeating Instrument :** It is provided with double vertical axis (two centres and two clamps) and two or more verniers reading to 10 to 5 seconds. It is used for measuring angles by repetition method.

2. **The Direction Instrument :** It has only one vertical axis and is provided with two or three micrometer microscope instead of verniers to read fractional parts of the angle smaller than the smallest division of the graduated circle. The horizontal circle of the instrument (25 or 30 cm) is usually graduated to 5 minutes and by means of micrometer microscopes an angle can be read directly to nearest second and 0.1 sec by estimation.

Double reading Type (such as zeiss, wild or Tavistock) theodolites with optical micrometer are being used in recent times in place of the theodolites of the micrometer type. These are very small, light, completely dust-proof and waterproof, and can be electrically illuminated. It has a horizontal circle 7-8 cm dia made of glass. The graduations are therefore very much fine. The

graduations of the diametrically opposite parts of the circle are brought together or slightly separated into the same field of view by means of a systems and prism of the readings can be observed from a single microscope. The error due to eccentricity of circle is automatically eliminated because the micrometers are so designed that the observed single reading gives the mean of the readings at the diametrically opposite parts of the circle.

3. **The Direct Scale Reading Instrument :** The most modern theodolite for measuring angles precisely is of direct scale reading type which eliminates the use of a micrometer or vernier. The horizontal and vertical circles of this instrument are figured continuously from 0^0 to 359^0 and are read through the single circle reading eye-piece lying alongside its main telescope. The reading by fine lines across the circles, is direct to 1 minute of arc and by estimation to $\frac{1}{10}$ th of this.

This instrument has permanent built-in optical plummet focusing from one metre to infinity and the plummet eye-piece can be seen just below the plate spirit level. The altitude spirit level is also built into the theodolite providing complete protection. It has a repetition clamp which is a single lever with two positions to allow the circle to be firmly secured to either the tribrach or to the alidade. The clamp also enables a subtense bar to be used. The instrument has only one tangent screw to perform the function of both tangent screws on a conventional type and its repetition clamp has replaced the function of the lower clamp.

Question 21. Write a short note on 'Gyro-theodolite'.

Answer : GYRO-THEODOLITE

A Gyro is an attachment mounted on a theodolite for determining true meridian at any place during day or night with a high degree of accuracy with a standard deviation of ± 20" in less than 20 minutes of working time.

Astronomical observations with a compass are liable to gross error due to uncertainties caused by local anomalies in the earth's magnetic field and it requires good weather, exact timing and lengthy calculations. Whereas the advantages gained by the use the gyro attachment are obvious.

The principle of gyro attachment is that when any gyroscope is suspended like a plumb bob, it constrained to lie in a horizontal plane and then spun. The rotation of the earth induces a couple causing the axis of the gyroscope to oscillate in a horizontal plane about the earth's axis of rotation which ultimately brings it towards the true north.

The rechargeable nickel-cadmium batteries and an electronic unit which converts 12 volts D.C. to 115 volts A.C., 400 cycles per seconds are connected with the gyro motor to run it. The gyro motor is suspended on a thin metal tape which hangs like a plumb bob. Its axis is always held horizontal. The theodolite on which the gyro mounted is levelled up. The gyro motor is spinning at about 22000 r.p.m. and trying to maintain its initial spinning space due to its moment of inertia. But it is pulled out of the original spinning plane by the earth's spinning motion as it is earthbound together with the tripod and theodolite. Its spin axis oscillating around the plumb line and ultimately taking its position in a meridian plane.

The gyro attachment is positioned in a theodolite bridge in such a way that the gyro's s axis and the line of sight of the telescope lie in the same vertical plane when the gyro mark is the zero position viewed through the gyro observation window.

To obtain the direction of the true north from the swinging gyro mark the following meth is applied :

Reversed point Method : As the gyro axis takes a long period to come to an equilibri position, the moving gyro mark is brought as sharply as possible in the middle of the scale means of horizontal slow-motion screw of theodolite. At the reversal point the gyro point appea to rest for a short time. The reading of ;the horizontal circle of the theodolite is noted just at th moment. In this manner three successive reversal points are observed and respective readin are noted from the horizontal circle.

The horizontal circle value for the true north are then calculated very easily using t Schuler's Mean Method.

Question 22. What are the sources of error in theodolite observations ? What precautions a taken to reduce them ?

Answer : SOURCES OF ERROR IN USING THEODOLITE, PRECAUTIONS

The sources of error in using a theodolite may be classified as :

1. Instrumental errors.
2. Personal errors.
3. Natural errors.

1. **Instrumental Errors :** these may be due to the imperfect adjustment of the instrume or of the structural defects of instrument itself. The sources of error and precautions required minimise them are -

(a) *Non-adjustment of plate levels* - The error due to this may be minimised by levelling t instrument carefully with reference to altitude spirit bubble.

(b) *Non-adjustment of line of collimation* - The error may be eliminated by taking face le and face right observations of the angle and taking the mean of the observed values.

(c) *Non-adjustment of horizontal axis* - The error is eliminated by making double fa observations of the angle and taking the mean of the two values.

(d) *Non-adjustment of altitude bubble tube axis* - Taking the mean of two observations the required angle, one with telescope normal and the other with the telescope inverted, w eliminate the error caused by such non-adjustment.

(e) *Index error* - Reading with verniers on both faces, and averaging the results, w automatically correct for index error.

(f) *Graduation error* - The angle should be measured many times on different parts of t horizontal circle and the mean should be taken to minimise the error.

(g) *Eccentricity of centres* - The error is eliminated by reading both verniers and averagi the two observations.

(h) *Eccentricity of verniers* - The error is eliminated by reading both verniers and averagi

the two values.

Instrumental error can be kept within reasonable limits if theodolite is maintained in good adjustment Damage or defective instrument which cannot be adjusted except by the maker should not be used.

2. **Personal Error :** These are mostly accidental errors and cannot be fully eliminated but can be kept within reasonable limits by care in levelling and observing-

(a) *Inaccurate centering* - The error is produced to a negligible amount by carefully setting up the instrument exactly over or under the station mark.

(b) *Inaccurate levelling* - The error is minimised by levelling the instrument carefully using the altitude bubble.

(c) *Slip* - To minimise error due to slip -

(i) The Instrument should be firmly secured to its tripod stand and tripod stand itself should be properly clamped.

(ii) Sliding plate for centering should be properly clamped.

(iii) Levelling screws should not be left loose.

(iv) Any clamp not in use should be sufficiently tightened.

(d) *Parallax* - This should be removed before any observations.

(e) *Displacement of station mark* - To guard against this the instrument station should be selected on firm ground and station should be marked with long peg or nail driven solidly into the ground.

(f) *Some other sources of Personal error are* -

(i) Manipulation of wrong tangent screw.

(ii) Poor focussing.

(iii) Station mark not properly bisected.

(iv) Placing hand on stand.

(v) Incorrect setting of vernier.

(vi) Incorrect reading.

(vii) Incorrect booking.

The error due to above causes may be minimised by properly bisecting the station mark at its bottom and correctly focussing the telescope. Habitual checks and alertness in work can only guard against the above sources of error.

3. **Natural Errors :** These may be due to the following :

(a) Wind, producing vibrations of the instrument.

(b) High temperature producing refraction and causing expansion of different parts of the instrument.

(c) Unequal settlement of tripod stand.

The above sources are not large enough to effect measurements appreciably. But precise

work should always be done under most favourable atmospheric condition avoiding field work during high winds and during mid-day in summer seasons.

Question 23. What improvements have been incorporated in the modern theodolites as compared with older models ?

Answer : IMPROVEMENTS IN MODERN THEODOLITES

The improvements that have been incorporated in the construction of modern theodolites are –

1. Very compact and light, made of specially selected materials giving lasting reliability in operation even under most unfavourable conditions.

2. Permanent adjustments are maintained for long period.

3. The introduction of transparent glass circles with opaque graduations has made it possible to reduce the diameter of circles and yet with increased accuracy, thereby reducing the size and weight of the instrument.

4. There is no loss of accuracy due to distortion or ageing since optical glass used is one of the most stable substances known. Another great advantage is that neither the circles nor the auxiliary scales are liable to tarnish.

5. In order to get reading smaller than the divisions on the circle, auxiliary scales or graticules have been introduced into the focal plane of the eye-pieces, where they are clearly defined and therefore easily read.

6. The reading eye-pieces are normally observed from the same stance as the main telescope and this saves time and reduces the risk of disturbances caused by walking around the instrument for observations.

7. Far reaching digitization of circle reading effectively prevents reading errors. Reading upto 7 figures is now possible without any counting or estimation. Graduated circles of some modern theodolites are divided into Centisimal System to express measured angles as decimals.

8. Short but high powered internal focusing telescopes are used in all modern optical theodolites, there by reducing the size and weight of the whole instrument.

9. High quality apochromatic telescope ensures accurate bisection of objects. Telescope lenses are coated by reflection reducing material and focussing rings or sleeves have been introduced on the telescope for focussing.

10. Illumination mirrors are provided to get clear reading from vertical and horizontal circles during day time. Artificial light arrangement help to get readings below ground or during night.

11. All circles, auxiliary scales, reading microscopes, etc. are totally enclosed. It is completely water proof and dust proof.

12. Interchangeable auto-collimation eye-piece is provided for any precise work with highest degree of accuracy.

13. Optical plummet device has been introduced for accurate centering even during stormy weather.

14. The vernier has been replaced by micrometer microscope.

15. Most modern theodolites are fitted with stadia diaphragm and with a telescope of suitable aperture, commonly 4 cms and a magnification of 25 to 30 diameters and whose additive constant is negligible.

16. In some modern instruments Alpha membrane has been fitted for tortionally rigid connection between tripod and instrument.

17. Detachable tribrach has been introduced for forced centering with a target.

18. Gyro attachment for determining true meridian and carrying handle for convenience have also been incorporated in some modern theodolites.

19. Automatic theodolites supplemented with 250 mm plotting table are now used to plot a scale picture of the surveyed site directly at the opencast mine.

20. Suspended type theodolite, a micro-optical instrument maybe suspended from a cantilever arm which is threaded into the leg of a timber set. The vertical axis of rotation of this instrument may be set plumb using the force of gravity of the instrument. A more accurate setting is done by means of two micrometers. Hanging or suspended type theodolite is specially suitable for highly inclined shafts or winzes or dense traffic workings.

21. One of the new developments in opencast mine surveying is the photogrammetic method (either ground stereophoto or aerial stereophoto survey) by a photo-theodolite which is a combination of a camera to photograph the object and a theodolite for the orientation of the camera.

WORKED OUT EXAMPLES

LENGTH AND BEARING OF CLOSING LINE

The following trigonometric relations are used in computing the unknowns :

Latitude = Plan distance x Cos. bearing.

Departure= Plan distance x Sine bearing.

These two simple equations maybe applied successively to all the bearings and distances in a survey in order to calculate the partial co-ordinates of each point. Note, however, that *the bearing referred to are quadrant bearings* and that, if the bearings are expressed in azimuths, they must be converted to quadrant bearings before the co-ordinates can be calculated in this

way. The quadrant in which the bearing occurs determines whether the calculated latitude is North or South, and the calculated departure East or West.

If the co-ordinates of a point, or of two points, with respect to the initial station are given, the corresponding bearing and distance can be calculated from the following general rules :

Tan angle of bearing = Departure / Latitude

Plan distance = $\sqrt{\text{Latitude}^2 + \text{Departure}^2}$

or Plan distance = Latitude x Secant bearing

or Plan distance = Departure x Cosec bearing.

CALCULATION OF AREA BY CO-ORDINATES

The area of any closed traverse may be accurately calculated direct from the co-ordinates without first plotting the survey. There are several methods of utilising the known co-ordinates for this purpose, but one of the accepted rules is as follows :

Multiply the total latitude of each station in turn by the algebraic sum of its partial departure and that of the station next following. Then the area is half the algebraic sum of all the products thus obtained.

In some cases, only the total co-ordinates, with respect to the initial point, are given, it is more convenient in such a case utilise the following rule :

Multiply the algebraic sum of adjacent total latitudes by the algebraic difference of adjacent total departures. Then the area is half the algebraic sum of all the product thus obtained.

EXAMPLES

Example 1. The co-ordinates of two points L and M are as under :

Points	North co-ord.	East co-ord.
L	405.50 m	630.60 m
M	755.50 m	310.60 m

Find the length and bearing of the line LM.

Answer : Latitude of LM = difference between North Co-ordinates

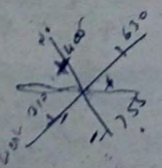
= 755.50 – 40.50 = 350.00

Departure of LM = Difference between East Co-ordinates

= 310.60 – 630.60 = - 320.00

$$\therefore \text{Tan}\,\theta = \frac{\text{Difference of deps.}}{\text{Difference of lats.}} = \frac{-320.00}{+350.00} = 0.9142$$

$\theta = 42°26'$ North-West $\begin{bmatrix}\text{Lat} & = + \text{ve} \\ \text{Dep} & = - \text{ve}\end{bmatrix}$



∴ Quadrant bearing of line LM = N 42°26'W

Length of line LM = $\sqrt{\text{Lat}^2 + \text{Dep}^2} = \sqrt{350^2 + 320^2} = 474.20$ m

Check : Length of line LM = Dep. of LM x Cosec θ

= 320.00 x Cosec 42°26' = 474.20 m.

Example 2. The co-ordinates of two points A and B, in relation to a common point or origin are as follows :

A	Latitude : North 188.6 m	Departure : East 922.4 m
B	Latitude : South 495.4 m	Departure : East 58.6 m

Calculate the co-ordinates of P which lies midway between A and B.

Answer : Latitude of P = $\frac{188.6 + 495.4}{2}$ – 188.6 = 154.4 m South

Departure of P = $\frac{922.4 - 58.6}{2}$ + 58.6 = 490.5 m East.

Example 3. The following traverse is run from A to E between which there occur certain obstacles :

Line	Dist. in m.	Bearing
AB	445	35°15'
BC	524	345°18'
CD	595	289°14'
DE	415	31°30'

It is required to peg the point midway between A and E. Calculate the length and bearing of a line from C to this midway point.

Answer : Assuming original values of station A are : Latitude = 0 metre, and departure = 0 metre, the co-ordinates of other stations are tabulated below -

Line / Stn.	Bearing θ	Distance in metres L	Partial Co-ordinates: Latitude L x Cosθ	Partial Co-ordinates: Departure L x Sinθ	Total Co-ordinates: Lat	Total Co-ordinates: Dep
A					0	0
AB	N35°15'E	445				
B			+363.40	+256.83	+363.40	+256.83
BC	N14°42'W	524				
C			+506.85	–132.97	+870.25	+123.86
CD	N 70°46'W	595				
D			+196.00	–561.79	+1066.25	–437.93
DE	N31°30'E	415				
E			+353.84	+216.83	+1420.09	–221.10

Let the midway point between A and E be F.

Latitude of point F = + 1420.09 ÷ 2 = + 710.05 m

Departure of point F = – 221.10 ÷ 2 = – 110.50 m

Bearing of line CF = $\tan^{-1}\frac{-110.50 - 123.86}{+710.05 - 870.25}$ = S 55°38'42" W

Length of line CF = [123.86 – (–110.50)] cosec 55°38'42"

= 234.36 cosec 55°38'42" = 283.86 metres.

Example 4. (a) Discuss the advantages of co-ordinate plotting over protractor plotting.

(b) Following are the results of booking in a traverse survey carried out underground from station A to E. The seam is level.

Line	Azimuth	Distance	Remarks
AB	170°	50 m	Level
BC	95°	57 m	Level
CD	50°	40 m	Level
DE	145°	66 m	Level

Calculate the azimuth and length of the roadway to be driven from E to A.

Answer : (a) *Advantages of co-ordinate plotting over protractor plotting are-*

1. It enables a survey to be plotted with great precision.
2. Each point may be plotted independently of any other point in the survey.
3. The bearing and length of the closing line may be calculated to any required degree of accuracy consistent with the accuracy of the instrumental work.
4. Accuracy can be checked before plotting a closed traverse.
5. The area of any closed traverse may be calculated directly from the known co-ordinates of the angular points.
6. Shrinkage of plan at any part can be ascertained readily from the co-ordinate graticules (or squares) which cover the plan with a net work of scale.
7. Plotting of large survey can proceed simultaneously on different sheets or on different parts of sheet.
8. The computation form is available for subsequent checking.

On the otherhand, plotting by protractor is a simple and convenient process, but only a limited degree of accuracy is obtainable depending on the scale of the plan, the smallest division on the protractor and the skill of the surveyor. Moreover, unless the survey is a closed one, no check on the plotting is possible, and if the bearing of the closing line is desired this can only be obtained approximately.

(b) **Calculation of Co-ordinates :**

Line	Azimuth	Quadrant Bearing θ	Distance L	Partial Co-ordinates Lititude L x Cosθ	Partial Co-ordinates Departure L x Sinθ	Station
AB	170°	S 10° E	50	–49.240	+8.682	B
BC	95°	S 85° E	57	–4.968	+56.783	C
CD	50°	N 50° E	40	+25.712	+30.642	D
DE	145°	S 35° E	66	–54.064	+37.856	E
Total Co-ordinates of E =				– 82.560	+133.963	

Length of the roadway EA = $\sqrt{82.560^2 + 133.963^2}$ = 157.36 m.

Bearing of EA = $\tan^{-1}\frac{+133.963}{-82.56}$ NW = 58°21'17"W

Azimuth = 360° - 58°21'17" = 301°38'43"

Example 5. Plot the following survey by co-ordinates and calculate the length and bearing of the closing line :

Line	Meridian Bearing	Distance in m.
AB	34°	400
BC	305°	250
CD	275° 30'	310
DE	225°	260
EF	148° 30'	395

Answer : Assuming the co-ordinates of station A to be latitude Om and departure Om, the co-ordinates of other stations are calculated and tabulated below -

Line / Stn.	Meridian Bearing	Quadrant Bearing 0	Distance in Metres L	Partial Co-ordinate Lat L x Cosθ	Partial Co-ordinate Dep L x Sinθ	Total Co-ordinate Lat	Total Co-ordinate Dep
A						0	0
AB	34°	N 34° E	400				
B				+331.615	+223.677	+331.615	+223.677
BC	305°	N 55° W	250				
C				+143.394	–204.788	+475.009	+18.889
CD	275°30'	N 84°30'W	310				
D				+29.712	–308.57	+504.721	–289.681
DE	225°	S 45° W	260				
E				–183.848	–183.848	+320.873	–473.529
EF	148°30'	S 31°30' E	395				
F				–336.793	+206.387	–15.92	–267.142
Total Co-ordinates of F =						–15.92	–267.142

Bearing of FA = $\tan^{-1}\frac{+267.142}{-15.92}$ = N 86°35'22"E

Length of FA = 267.142 Cosec 86°35'22" = 267.616 m.

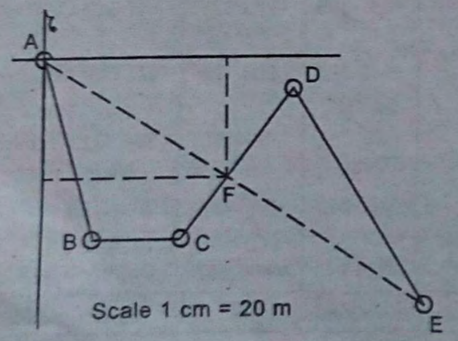
Example 6. The following are the notes of an underground traverse in level seam :

Line	Azimuth	Distance
AB	170°	400 m
BC	95°	200 m
CD	50°	420 m
DE	145°	550 m

Calculate the azimuth and length of the roadway to be driven to connect the stations E and A. Check your answer by plotting on a scale of 1 in 2000.

Answer : Assuming the co-ordinates of station A to be latitude = 0 metre and departure = 0 metre, the co-ordinates of other stations are tabulated below -

Line / Station	Azimuth	Quadrant Bearing θ	Horizontal Distance L	Partial Co-ordinates		Total Co-ordinates	
				Lat L x Cosθ	Dep L x Sinθ	Lat	Dep
A						0	0
AB	170°	S10°E	400				
B				–393.923	+69.459	–393.923	+69.459
BC	95°	S 85°E	200				
C				–17.431	+199.239	–411.354	+268.698
CD	50°	N 50°E	420				
D				+269.971	+321.738	–141.383	+590.436
DE	145°	S 35°E	550				
E				-450.533	+315.467	–591.916	+905.903



Length of the new roadway EA

= $\sqrt{(-591.91)^2 + (905.90)^2}$

= 1082.137 metres.

Bearing of EA

= N ($\tan^{-1}\frac{905.90}{591.91}$)W

= N 56°50'22" W

Azimuth of EA

= 360° – 56°50'22"

= 303°09'38"

Check the calculated length and bearing by co-ordinates plotting from total co-ordinates of each station on a scale of 1 : 2000.

Example 7. The following are the notes of an underground traverse :

Line	Quadrant bearing	Hor. Distance
AB	N 36°14'E	320 m
BC	N 6°18'W	420 m
CD	S 81°44'E	115 m

The co-ordinates of A are 443.7 m South and 186.1 m West based on a local origin. Calculate the co-ordinates of the station B,C, and D, and determine the length and bearing of closing line DA.

Answer : The total co-ordinates of the station B,C, and D are calculated and tabulated below -

Stn.	Line	Quadrant Bearing	Dist	Partial Co-ordinate		Total Co-ordinate	
				Lat	Dep	Lat	Dep
A						-443.7	-186.1
	AB	N 36°14'E	320				
B				+258.118	+189.144	-185.58	-3.044
	BC	N 6°18'W	420				
C				+417.463	-46.08	+231.88	+43.04
	CD	S 81°44'E	115				
D				-16.53	+113.80	+215.35	+70.76

Bearing of DA = $\tan^{-1}\frac{\text{Dep. A} - \text{Dep. D}}{\text{Lat. A} - \text{Lat. B}}$

= $\tan^{-1}\frac{(-186.1) - (70.76)}{(-443.7) - (215.35)}$

= $\tan^{-1}$ 0.3897

Bearing of DA = 21°17'35" SW

Length of DA = Difference in latitudes x secθ

= 659.05 x Sec 21°17'35" = 707.33 metres.

Example 8. The following are the notes of underground survey :

Line	Azimuth	Dist. in metres
AB	333°	200
BC	62°	210
CD	118°	190
DE	197°	200

Calculate the length and bearing of the drivage EA. Plot the resutlts on a suitable scale.

Answer : The co-ordinates of the stations B, C, D and E are calculated and tabulated below-

Line Stn.	Azimuth	Quadrant Bearing θ	Dist L	Partial Co-ordinates	
				Lititude L x Cosθ	Departure L x Sinθ
AB B	333°	N 27°W	200	+178.20	–90.80
BC C	62°	N 62°E	210	+98.59	+185.42
CD D	118°	S 62°E	190	–89.20	+167.76
DE E	197°	S 17°W	200	–191.26	–58.47
Total Co-ordinates of E =				–3.67	+203.91

Bearing of line AE = $\tan^{-1} \frac{+203.91}{-3.67}$ = S 88°58'08" E

Hence bearing of drivage EA = N88°58'08"W

Length of the drivage EA = 3.67 Sec 88°58'08" = 203.94 m.

Example 9. The survey notes of a traverse are given below :

Line	Azimuth Bearing	Horizontal Distance
AB	44°	120m
BC	329°	126m
CD	257°	147m
DE	174°	79.5m
EF	260°	105m

Calculate the co-ordinates of the stations, to a local origin at A, and determine the lengt of the closing line.

Answer : The latitude and departure of each line or draft are calculated and the various figure are set out in the table -

Line Stn.	Azimuth Bearing	Quadrant Bearing θ	Hor. Dist L metre	Partial Co-ordinates		Total Co-ordinates	
				Latitude L x Cosθ	Departure L x Sinθ	Latitude	Departure
A						0	0
AB B	44°	N 44°E	120	+86.320	+83.359	+86.320	+83.359
BC C	319°	N 31°W	126	+108.00	–64.894	+194.320	+18.465
CD D	257°	S 77°W	147	–33.067	–143.232	+161.253	–124.767
DE E	174°	S 6°E	79.5	–79.064	+8.3100	+82.189	–116.457
EF F	260°	S 80°W	104	–18.233	–103.401	+63.956	–219.858

Length of the closing line = $\sqrt{Lat^2 + Dep^2}$

= $\sqrt{(63.956)^2 + (-219.858)^2}$

= 228.97 metres.

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Example 10. Following are the notes of an underground traverse survey made to ascertain whether the position of point F is in alignment with the BA extended :

Line	Azimuth	Distance in m.
AB	352°30'	156
AC	122°	83
CD	207°	119
DE	127°	129
EF	210°	143

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Calculate the difference in azimuth of line AF and BA and also the distance AF.

Answer : SURVEY TO ASCERTAIN THE POSITION OF A POINT

The azimuths are converted to quadrant bearings and then the co-ordinates are calculated as shown in the table.

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Line	Bearing	Distance	Partial Latitude	Partial Departure	Total Latitude	Total Departure	Station
					0.0	0.0	A
AC	122° S 58°E	83m	–43.98	+70.39	–43.98	+70.39	C
CD	207° S 27°W	119m	–106.03	–54.03	–150.01	+16.36	D
DE	127° S 53°E	129m	–77.63	+103.03	–227.64	+119.39	E
EF	210° S 30°W	143m	–123.84	–71.50	–351.48	+47.89	F

Co-ordinates of F are : Latitude = –351.48 and Departure = + 47.89

Bearing of line AF = $\tan^{-1}\frac{+47.89}{-351.48}$ = S 7°45'32"E

Bearing of line BA = 352°30' - 180° = 172°30' = S 7°30'E

Difference of bearings of AF and BA = 15'32"

Distance AF = $\sqrt{351.48^2 + 47.89^2}$ = 354.73 metres.

Example 11. Plot the undernoted azimuth bearings by co-ordinates on a scale of 1 cm to 100 metres, and calculate the distance between A and B.

From A.	30° (NE)	150 m	rising 1 in 4
	90°	400 m	Level
	135°	380 m	dipping 1 to 3
	240°	210 m	level
	330°	185 m	level to B.

Answer : Reduction of inclined distances to plotting distances -

150 m at 1 in 3 = $\frac{150 \times 3}{\sqrt{10}}$ = 142.30 m plan distance

380 m at 1 in 4 = $\frac{380 \times 4}{\sqrt{17}}$ = 368.65 m plan distance

Booking of survey and Co-ordinates :

	Azimuth	Q.B. θ	Hor. Distance L	Partial Co-ordinates		Total Co-ordinates	
				Lat L x Cosθ	Dep L x Sinθ	Lat	Dep
From A	30°	N 30°E	142.30	+123.23	+71.15	+123.23	+71.15
	90°	Due East	400		+400	+123.23	+471.15
	135°	S 45°E	368.65	-260.67	+260.67	-137.44	+731.82
	240°	S 60°W	210	-105	-181.86	-242.44	+549.96
To B	330°	N 30°W	185	+160.21	-92.5	-82.23	+457.46

The plan distance from A to B = $\sqrt{Lat^2 + Dep^2}$

= $\sqrt{(82.23)^2 + (457.46)^2}$

= $\sqrt{216031}$ = 464.79 m.

Example 12. Undernoted are details of a short traverse which was made for the purpose of determining the bearing and length of a level cross measure drift, proposed to be driven from the point B to another point vertically under the centre of the shaft A, with which it is to be connected by piercing upwards.

Calculate, as accurately as the undernoted tables will allow, the bearing and distance from the point B to the starting point of the proposed shaft, and the extent of the shaft to be pierced up.

From centre of shaft A -	East	195 m	Level
	S 9°E	621 m	dipping 33°
	S 8°E	554 m	dipping 26°
	N 87°W	371 m	Level to B.

Answer : Reduction of inclined distance to plotting distances :

621 m dipping 33° = 621 x cos 33° = 520.81 m plotting distances :
and 621 x sin 33° = 338.22 m difference in level.

554 m dipping 26° = 554 x cos 26° = 497.93 m plotting distance.
and 554 x sin 26° = 242.86 m difference in livel.

Calculation of Co-ordinates :

Latitude 520.81 x Cos 9° = 514.40 m
497.93 x Cos 8° = 493.08 m
371.00 x Cos 87° = 19.42 m.

Departure 520.81 x Sin 9° =81.47 m
497.93 x Sin 8° = 69.30 m
371.00 x Sin 87° = 370.49 m.

Booking of Survey Details :

Bearing	Distance	Latitude		Departure	
		N	S	E	W
From A. East	195.00	-	-	195	-
S 9°E	520.81	-	514.40	81.47	
S 8°E	497.93	-	493.08	69.30	
To B. N87°W	371.00	19.42	-	-	370.49
	Total	19.42	1007.48	345.77	370.49

$$\text{Tangent of Bearing} = \frac{\text{Difference in departure}}{\text{Difference in latitude}}$$

$$= \frac{370.49 - 345.77}{1007.48 - 19.42} = \frac{24.72}{988.06} = 0.025$$

∴ Bearing is N 1°26'E from B to A.

Distance is latitude x Secant of 1026' = $988.06 \times \frac{1}{0.999} = 989.04$

Extent of shaft to be pierced upward = 338.22 + 242.86 = 581.08 metres.

Example 13. Following are the notes of a dial traverse carried out from A to E.

Line	Azimuth	Hor.Distance	Inclination
AB	210°	60 m	10° dipping
BC	110°	90 m	15° dipping
CD	60°	100 m	12° dipping
DE	130°	120 m	0° level

Calculate the azimuth, the length and inclination of the roadway to be driven from E to A

Answer : Vertical fall = Hor. dis. x tan angle dip

Line AB verrtical fall = 60 x tan 10° = 10.580 m

Line BC vertical fall = 90 x tan 15° = 24.115 m

Line CD vertical fall =100 x tan 12° = 21.254 m

Line DE vertical fall = Level = 0 m

Total fall from A to E = 55.949 m

Co-ordinates Calculations :

Line Station	Azimuth	Quadrant Bearing θ	Horizontal Dist L	Partial Co-ordinates	
				Latitude L x Cosθ	Departure L x Sinθ
AB	210°	S 30°W	60		
B				–51.962	–30.00
BC	110°	S 70°E	90		
C				–30.781	+84.573
CD	60°	N 60°E	100		
D				+50.000	+86.602
DE	130°	S 50°E	120		
E				–77.134	+91.924
Total Co-ordinates E =				–109.877	+233.099

Bearing of EA = $\tan^{-1}\frac{233.099}{-107.877}$ NW = N 64°45'43"W

Azimuth of the roadway EA = 360° - 64°45'43" = 295° - 14' - 17"

Length of EA = 233.099 x cosec 64°45'43" = 257.70 m

Inclination of Line EA = 55.949 in 257.70 = 1 in 4.60.

Example 14. It is proposed to connect two points A and B by driving galleries from either point. A traverse is run from A to B, and the following bookings are obtained :

Line	Magnetic Bearing	Horizontal Length	Gradient
AC	175°	150 metres	1 in 5 dip
CD	85°	306 "	Level
DE	45°	135 "	1 in 8 rise
EB	305°	345 "	1 in 10 rise

Calculate the length, bearing and gradient of line AB.

Answer : The co-ordinates of the stations are calculated –

Line Stn.	Azimuth	Quadrant Bearing θ	Hor. Length in metres L	Latitude L x Cosθ	Departure L x Sinθ
AC C	175°	S 5°E	150	–149.430	+13.074
CD D	85°	N 85°E	306	+26.670	+304.830
DE E	45°	N 45°E	135	+95.459	+95.459
EB B	305°	N 55°W	345	+197.880	–282.610
Total Co-ordinates of B				+170.579	+130.753

Bearing of line AB = $\tan^{-1}\frac{130.753}{170.579}$ = N 37°28'15"E

Horizontal length of AB = 170.579 sec 37°28'15" = 214.93 m.

Fall from A to C = 150 ÷ 5 = -30 m

Rise from D to B = 135 ÷ 8 = +16.875 m

Rise from E to B = 345 ÷ 10 = +34.500 m

Rise from A to B = +21.375 m

Gradient AB = 21.375 m vertical in 214.93 m horizontal

= 1 to 10 (approx) rising.

Inclined length of AB = $\frac{214.93 \times \sqrt{101}}{10}$ = 216.00 m.

Example 15. (a) What are the different ways of calculation of area of a polygon with given total co-ordinates ?

(b) Following are the total latitudes and departures of the vertices of a polygon in metres. Calculate the area of the polygen in hectares.

STATION	TOTAL LAT. IN M.	TOTAL DEP. IN M.
A	+120.0	+143.2
B	+ 49.3	+213.9
C	– 80.6	+138.9
D	– 80.6	+ 70.6
E	– 20.0	– 30.0

Answer : CALCULATION OF AREA FROM TOTAL CO-ORDINATES

When the total co-ordinates, with respect to the initial point, are given, the area of a polygon may be calculated by utilising any of the following rules :

1. Calculate the partial departures from the given total departures. Multiply the total latitude of each station in turn by the algebraic sum of its partial departure and that of the station next following. Then the area is half the algebraic sum of all the products thus obtained.

2. Multiply the algebraic sum of adjacent total latitudes by the algebraic defference of adjacent total departures. Then the area is half algebraic sum of all the product thus obtained.

In both the above two methods, the words latitude and departure maybe interchanged without in any way altering the final result.

It sometimes happens that the total co-ordinates of a number of stations are given, but the point of origin does not coincide with one of the stations. In such a case, it is convenient to reduce the co-ordinates to one of the stations as origin.

(d) **Calculation of Area :** Assuming A to be the new point of origin, all the latitudes are corrected by the algebraic addition of (-120) and the departures by the algebraic addition of (-143.2). The corrected co-ordinates are now –

	Latitudes	Departure
A	+120 + (–120) = 0	+143.2 + (–143.2) = 0
B	+49.3 + (–120) = –70.7	+213.9 + (–143.2) = +70.7
C	–80.6 + (–120) = –200.6	+138.9 + (–143.2) = –4.3
D	–80.6 + (–120) = –200.6	+70.6 + (–143.2) = –72.6
E	+20.0 + (–120) = –100.0	–30.0 + (–143.2) = –173.2

Station	Total Latitude	Total Departure	Sum of Latitudes	Difference of Deps.	Double Area Sq. m.
A	0	0	-70.7	-70.7	+4998.5
B	-70.7	+70.7	-271.3	+75.0	-20347.5
C	-200.6	-4.3	-401.2	+68.3	-27402.0
D	-200.6	-72.6	-300.6	+100.6	-30240.4
E	-100.0	-173.2	-100.0	-173.2	+17320.0
A	0	0			

Double Area = 55671.4

Area = 55671 ÷ 2 = 27836 Sq.m. = **2.7836 Hectares.**

Example 16. Calculate the co-ordinates of the following survey, determine the bearing and length of the closing draft, and calculate the area of the polygon :

Line	Bearing	Distance
AB	N35°23'W	38.70m
BC	N13°42'E	80.70m
CD	N73°16'E	94.20m
DE	S43°19'E	57.90m
EF	S15°48'W	60.00m

Answer : The co-ordinates of F with respect to origin A –

Line	Q.B. θ	Distance in mts. L	Partial Co-ordinates		Total Co-ordirates		Station
			Lat L Cosθ	Dep L Sinθ	Latitude	Departure	
					0	0	A
AB	N35°23'W	38.70					
			+31.551	-22.410	+31.551	-22.410	B
BC	N13°42'E	80.70					
			+78.405	+19.113	+109.956	-3.297	C
CD	N73°16'E	94.20					
			+27.123	+90.210	+137.079	+86.913	D
DE	S43°19'E	57.90					
			-42.126	+39.720	+94.953	+126.633	E
EF	S15°48'W	60.00					
			-57.732	-16.338	+37.221	+110.295	F

Check : Total Co-ord. F +37.221 +110.295

Bearing of closing line FA :

$$\tan\theta = \frac{\text{Total departure}}{\text{Total latitude}} = \frac{110.295}{37.221}$$

$$\theta = 71°21'$$

∴ Bearing of FA = S 71°21'W

Length of FA = Total latitude x Secθ

= 37.221 x Sec 71°21' = 116.394 metres.

AREA OF POLYGON :

The following rule is adopted to calculate the area of the polygon :- Multiply the total latitudes of each station by the algebraic sum of the partial departures of two adjacent drafts. Half the algebraic sum of these products gives the area required.

The calculations are tabulated as follows -

Station	Total Latitude (L)	Partial Departure	Partial dep. of adjacent draft	Algebraic sum of partial deps (S)	L x S Sq. m.
A	0	0	-22.410	-22.410	0
B	+31.551	-22.410	+19.113	-3.297	-104.02
C	+109.956	+19.113	+90.210	+109.323	-11983.00
D	+137.079	+90.210	+39.720	+129.930	+17813.00
E	+94.953	+39.720	-16.338	+23.382	+2220.10
F	+37.221	-16.338	0	-16.338	-608.11

Double area = Algebraic sum = +30403.97

$$\text{Area} = \frac{30403.97}{2} = 15201.985 \text{ Sq. m.}$$

Example 17. Undernoted are the details of a closed traverse :

Line	Bearing	Distance
AB	N 80°E	439m
BC	Due South	488m
CD	S 60°W	377m
DA	N 10°W	609.5m

Calculate the area of traverse ABCD by co-ordinates. Tabulate your calculation in proper form.

Answer : Calculations of Co-ordinates –

Line Stn.	Quadrant Bearing θ	Length L Metres	Partial Co-ordinates		Total Co-ordinates	
			Latitude L x Cosθ	Departure L x Sinθ	Lat	Dep
A					0	0
AB	N 80°E	439				
B			+76.24	+432.33	76.24	+432.33
BC	South	488				
C			-488.00	0.00	-411.76	+432.33
CD	S 60°W	377				
D			-188.50	-326.50	-600.26	+105.83
DA	N 10°W	609.5				
A			+600.26	-105.83	0.00	0.00

Calculation of Area-

Station	Total Lat	Total Dep	Sum of Adjacent Lats	Difference of Adjacent Deps	Double Area Sq.M.
1	2	3	4	5	4 x 5
A	0.00	0.00	+ 76.24	-432.33	-32960
B	+ 76.24	+ 432.33	- 335.52	0.00	-
C	-411.76	+ 432.33	-1012.02	+ 326.50	- 330424
D	-600.26	+ 105.83	- 600.26	+105.83	- 63525

Double Area Total =- 4,26.909 Sq.m.

Area = 426909 ÷ 2 = 213454 Sq.m.

Example 18. Total co-ordinates of 3 points are as under –

A	N 1000m	E 300m
B	N 200m	W 50m
C	S 500m	E 1000m

A level seam of coal 15m thick is lying within the property of ABC, Calculate- (a) Area in acres, and (b) Tonnage of coal available. Assume the specific gravity of coal = 1.25.

Answer : CALCULATION OF AREA AND COAL AVAILABLE

(a) **Calculation of Area :** It is more convenient to utilise the rule '*sums of total latitudes and differences of total departures*' when only the total co-ordinates, with respect to the initial point are given :

Station	Lat	Deps	Sum of Adjacent lats	Difference of Adjacent Deps	Double Area Sq.m.
A	+ 1000	+ 300	+ 1200	+ 350	+ 420,000
B	+ 200	- 50	- 300	- 1050	+ 315,000
C	- 500	+ 1000	+ 500	+ 700	+ 350,000

Double Area = 1085,000

Hence area of the property = 1085000 ÷ 2 = 542500 Sq. m.

= 542500 ÷ 4047 = 134.05 Acres.

(b) **Total Coal Available** = 542500 x 15 x 1.25 = 10171875 tonnes.

Example 19. The co-ordinates, in metres of five stations, A,B,C,D and E, forming a polygon are as follows :

Station	Latitude	Departure
A	0	0
B	220 N	40 W
C	600 N	220 E
D	450 N	400 E
E	200 N	420 E

Calculate the area of the figure ABCDE to square metres.

Answer : Calculations are tabulated as under-

Station	Lat (mts)	Dep (mts)	Dep.moved up one line	Dep moved down one line and sign changed	Σ of Columns 4 and 5	Double area in square metres (Column 2 x 6)	
						÷	–
A	0	0	- 40	- 420	- 460		0
B	+220	-40	+220	0	+220	48400	
C	+600	+220	+400	+40	+440	264000	
D	+450	+400	+420	-220	+200	90000	
E	+200	+420	0	-400	-400		80000
				Totals =		+402400	-80000

∴ Double area = 322400 Sq. m.

Area of polygon ABCDE = 322400 ÷ 2 = 161200 Sq. m. **Ans.**

Example 20. The following latitudes and departures of the sides of a traverse ABCD are given. Calculate the missing bearing of BC and the length of CD.

Line	Reduced Bearing	Distance (metres)	Latitude		Deprature	
			N	S	E	W
AB	N 50°E	246	158.12		188.44	
BC	?	315	?		?	
CD	N 25°W	?	?			?
DA	S 17°44'W	678.6		646.35		206.69

Answer : The unknown readings may be obtained by solving the triangle BCD formed by affected sides BC and CD and the closing line BD.

ΣL = - 646.35 + 158.12 = - 488.23

$\Sigma D = -206.69 + 188.44 = -18.25$

$\therefore$ Co-ordinates of BD are (+488.23) and (+18.25)

BD is in the first quadrant as latitude and departure are positive

Let θ be the reduced bearing of BD.

$$\tan\theta = \frac{18.25}{488.23} \; ; \quad \text{or } \theta = 2^\circ 8'$$

R.B. of BD = N 2°8'

Length of BD = 488.23 Sec 2°8' = 488.4 meters.

In triangle BCD, angle CDB = R.B. of DC + R.B. DB.

$= 25^\circ + 2^\circ 8' = 27^\circ 8'$

Length BD = 488.4 m. and BC = 315 m.

$$\text{By sine rule, } \sin BCD = \frac{BD}{BC} \text{ Sin CBD}$$

$$= \frac{488.4}{315} \sin 27^\circ 8' = 0.7071$$

$\therefore$ Angle BCD = $45^\circ + 90^\circ = 135^\circ$

Angle DBC = 180° - angles (BCD + CDB)

$= 180^\circ - (135^\circ + 27^\circ 8') = 17^\circ 52'$.

$\therefore$ R.B. of BC = R.B. of BD + angle DBC

$= 2^\circ 8' + 17^\circ 52' =$ N 20°E.

$$\text{Length of CD} = BD \times \frac{\text{Sin DBC}}{\text{Sin BCD}} = \frac{488.4 \text{ Sin } 17^\circ 52'}{\text{Sin } 135^\circ} = 212.1 \text{ m}$$

Example 21. The folowing are notes of a traverse survey in a level seam :

Line	Azimuth	Distance
AB	170°	120m
BC	95°	60m
CD	50°	126m
DE	145°	165m

Calculate the azimuth and length of a new roadway to be driven on the centre line A-
Thereafter plot the survey on a scale of 1 cm. = 20 m and , by measurement on the plotting, sta
the co-ordinates, relative to A as zero, of a point F in the line CD from which two additional plac
may be set off to enable the driving of the new road to proceed simultaneously from A, F and

Answer : The co-ordinates of each station are calculated and tabulated as under :

Line Stn.	Azimuth	Q.B.	Distance mts. L	Partial Latitude (L cosθ)	Partial Departure (L sinθ)	Total Co-ordinates Latitude	Total Co-ordinates Departure
A						0	0
AB	170°	S 10°E	120				
B				-118.17	+20.835	-118.17	+20.8355
BC	95°	S 85°E	60				
C				-5.2288	+59.759	-123.398	+80.594
CD	50°	N 50°E	126				
D				+80.984	+96.516	-42.414	+177.110
DE	145°	S 35°E	165				
E				-135.15	+94.624	-177.564	+271.734

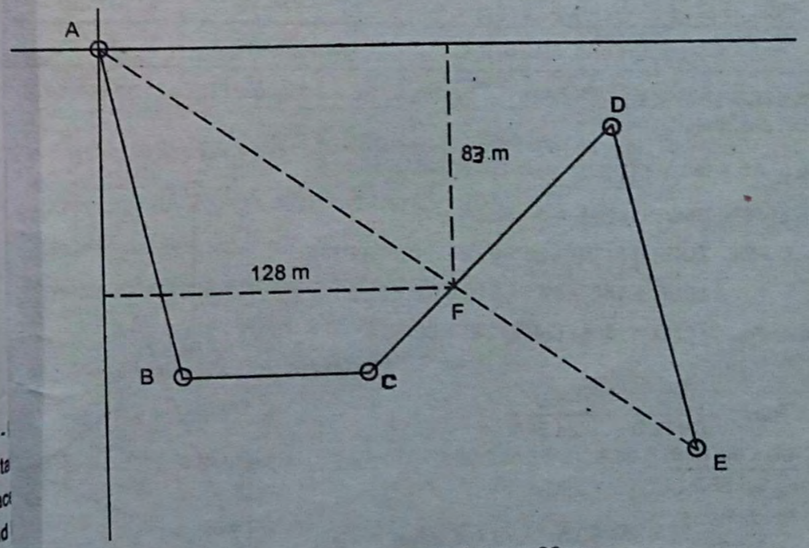
Length of new roadway EA $= \sqrt{(-177.564)^2 + (271.734)^2}$

$= 324.60$ m.

$$\text{Bearing of EA} = N\left(\tan^{-1}\frac{271.734}{177.564}\right) W = N\ 56^\circ 50'14''W$$

Azimuth of EA = 360° - 56°50'14" = 303°09'46"

PLOTTING



Scale 1 cm = 20 m

From the plotting the co-ordinates of F are 83 m South and 128 m East.

Example 22. The following are the notes of a theodolite traverse taken along the underground road-ways from A to E. It is proposed to join A E by means of a straight roadways. Find by calculation the position of point F on line CD from which drivage could be started towards A and E to effect quick connection.

Line	Azimuth	Distance
AB	271°30'	400 Mts.
BC	184°48'	550 Mts.
CD	120°00'	490 Mts.
DE	231°30'	781 Mts.

Answer :

Line / Station	Azimuth	Quadrant Bearing	Distance Mts.	Partial Co-ordinates: Latitude	Partial Co-ordinates: Departure
AB	271°30'	N 88°30'W	400		
B				+10.47	-399.86
BC	184°48'	S 4°48'W	550		
C				-548.60	-46.02
CD	120°00'	S 60°E	490		
D				-245.01	+424.35
DE	231°30'	S 51°30'W	781		
E				-486.18	-611.20
		Total Co-ordinates of E =		-1268.78	-632.73

The traverse is roughly plotted. From the bearing of AE the triangle FDE can be solved and the distance DF calculated.

Bearing of AE = $\tan^{-1}\frac{-632.73}{-1268.78}$ = S 26°30'22" W

Bearing of FE = Bearing of AE = S 26°30'22"W

In triangle FDE, ∠DEF = 51°30' - 26°30'22" = 24°49'38"

∠FDE = 180° - (60° + 51°30") = 68°30"

Therefore, ∠EFD = 180° - (24°59'38" + 68°30')

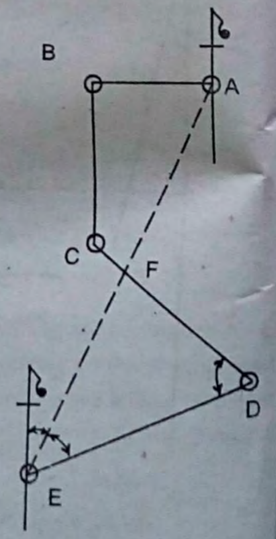
= 16°30'22"

By Sine Rule : $\frac{FD}{\text{Sin FED}} = \frac{ED}{\text{Sin EFD}}$

$\therefore FD = \frac{ED \times \text{Sin FED}}{\text{Sin EFD}}$

$= \frac{781 \times \text{Sin } 24°59'38"}{\text{Sin } 86°30'22"} = 330.60$ m.

Position of F is 330.60 m frp, D towards C.



Example 23. The following are notes of an underground traverse survey between the faces of headings BA and DE which are being driven to meet.

Determine by calculation, the distances from A and E, respectively to the point of intersection of the centre lines of the headings BA and DE.

Line	Azimuth	Distance
AB	80°	81.18 m
BC	180°	146.40 m
CD	240°	113.10 m
DE	350°	95.40 m

Answer : The traverse survey is roughly plotted and if X is the point of intersection of the centre lines of the headings BA and DE, we have to calculate the distances AX and EX.

Line Stn.	Distance mts L	Azimuth	Quadrant Bearing θ	Partial Latitude (L cosθ)	Partial Departure (L sinθ)	Total Co-ordinates: Latitude	Total Co-ordinates: Departure
A						0	0
AB	81.18	80°	N 80°E				
B				+14.096	+79.946	+14.096	+79.946
BC	146.40	180°	Due South				
C				-146.400	-	-132.304	+79.946
CD	113.10	240°	S 60°W				
D				-56.546	-97.947	-188.850	-18.000
DE	95.40	350°	N 10°W				
E				+93.951	-16.566	-94.899	-34.566

Bearing of line EA = $\tan^{-1}\frac{34.566}{94.899}$ NE = N 20°01'E

Length of EA = $\sqrt{(34.566)^2 + (94.899)^2}$ = 100.998 m

In triangle AEX,

Angle XEA = 10° + 20°01' = 30°01'

XAE = 80° - 20°01' = 59°59'

AXE = 180° - (30°01' + 59°59') = 90°

and EA = 100.998 m.

Now, $\frac{AX}{AE}$ = Sin 30°01' ∴ XA = 100.998 x Sin 30°01' = 50.524 m

and $\frac{EX}{AE}$ = Cos 30°01' ∴ EX = 100.998 x Cos 30°01' = 87.452 m.

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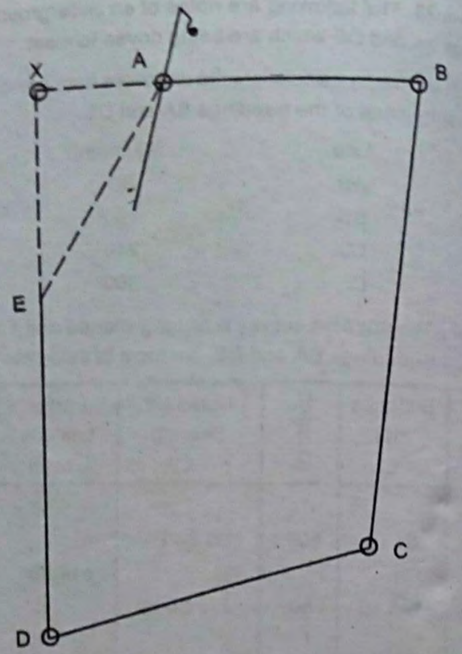
Therefore, the distance from A and E, respectively, to the point of intersection of the centre lines of the headings BA and DE are 50.524 and 87.452 metres.

Example 24. The following are the notes of an underground traverse :

Line	Azimuth	Hor. distance
AB	150°07'	144.510m
BC	187°58'	54.105m
CD	192°31'	82.911m
DE	188°34'	95.415m

It has been decided to drive a straight road from A to E, starting at A and simultaneously from X, midway between A and E. Calculate the length and bearing of a road to be driven from C to X.



Answer :

Line Stn.	Azimuth	Quadrant Bearing θ	Distance mts. L	Partial Latitude (L cosθ)	Partial Departure (L sinθ)	Total Co-ordinates	
						Latitude	Departure
A						0	0
AB	150°07'	S 29°53'E	144.510				
B				-125.295	+72.000	-125.295	+72.000
BC	187°58'	S 07°58'W	54.105				
C				-53.630	-7.500	-178.925	+64.500
CD	192°31'	S 12°31'W	82.911				
D				-80.940	-17.970	-259.865	+46.530
DE	188°34'	S 08°34'W	95.415				
E				-94.350	-14.214	-354.215	+32.316

Total co-ordinates of X are –

$$\text{Latitude} = \frac{-354.215}{2} = \text{S } 177.075 \text{ m}$$

$$\text{Departure} = \frac{+32.316}{2} = \text{E } 16.158 \text{ m}$$

Bearing of CX :

$$\tan\theta = \frac{(64.500) - (16.158)}{(-178.925) - (-177.084)} = \frac{48.342}{-1.841} = \tan 87°52'29''$$

Bearing of CX = N 87°52'29"W

Length of CX = (64.500 - 16.158) cosec 87°52'29" = 48.342 x cosec 87°52'29" = 48.375 metres

Example 25. The following are the notes of a survey made from the centre of shaft A to a survey station D in a coal seam :

Line	Azimuth	Distance	Vertical angle
AB	220°30'	150 m	+6°00'
BC	90°00'	187.5 m	Level
CD	222°05'	216 m	-14°30'

A cross-measure drift is to be driven from the point D to another point E in a overlying seam. Calculate the azimuth and horizontal length of drit DE, the co-ordinates of A and E are as follows.

A	N 420m,	W 24m
E	S 24m,	E 18m

Answer : The co-ordinates of stations are tabulated below :

Line Stn.	Azimuth Q.B. (θ)	Slope Distance d	Vertical angle α	Hor. Dist L d cosα	Partial Co-ordinate		Total Co-ordinates	
					Lat. L cosθ	Dep. L sinθ	Lat	Dep.
A							+420	-24
AB	220°30' S 40°40'W	150	+6°00'	149.18				
B					-113.44	-96.88	+306.56	-120.88
BC	90°00' Due East	187.5	Level	187.5				
C						+187.5	+306.56	+ 66.62
CD	222°05' S 42°05'W	216	-14°30'	209.12				
D					-155.20	-140.15	+151.36	-73.53

Co-ordinates of D = 151.36N. 73.53W

Co-ordinates of E = 24S, 18E

Algebraic difference of latitudes = (151.36) – (–24) = 175.36

Algebraic difference of departures = (–73.53) – (18) = 91.53

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Horizontal length of DE = $\sqrt{(91.53)^2 + (175.36)^2}$ = 197.80 metres.

Bearing of DE = S $\left(\tan^{-1}\frac{91.53}{175.36}\right)$ E = S 27°33'45"E.

Example 26. The following traverse notes refer to an u/g survey :-

Line	Azimuth	Hor distance
AB	188°13'	171.6m
BC	72°33'	190.5m
CD	324°20'	120.6m
DA	279°18'	88.5m

Calculate the error in the survey and then express it as a fraction of the total length of survey.

Answer : Assuming original values of station A are latitude O metre, departure O metre.

Line Stn.	Distance mts. L	Azimuth	Quadrant Bearings θ	Partial Latitude (L cosθ)	Partial Departure (L sinθ)	Total Co-ordinates	
						Latitude	Departure
A						0	0
AB	171.6	188°13'	S8°13'W				
B				-169.830	-24.525	-169.830	-24.525
BC	190.5	72°33'	N72°33'E				
C				+57.124	+181.720	-112.706	+157.195
CD	120.6	324°20'	N 35°40'W				
D				-97.980	-70.320	-14.726	+86.875
DA	88.5	279°18'	N80°42'W				
A				+14.301	-87.335	-0.425	-0.460

571.2 metres = Total length

Error in latitude = 0.425

Error in departure = 0.460

Bearing od closing line = $\tan^{-1}\frac{\text{difference in dep.}}{\text{difference in lat.}} = \tan^{-1}\frac{0-(-0.460)}{0-(-0.425)}$

=47°16'NE

Length of line = 0.425 x sec 47°16' = 0.62627 metre.

Total length of traverse line = 571.2 metres.

and, **Error** = 0.62627 metre.

Hence accuracy = $\frac{0.62627}{571.2} = \frac{1}{912.22}$

Example 27. The co-ordinates of two points P and Q are as given below.

P	193N	330E
Q	346N	502E

The megnetic bearing to an inaccessible point R is 140°30' from P and 183°30' from Q. Calculate the co-ordinates of the point R.

Answer : Bearing of PR = 140°30' = S 39°30'E

Bearing of RQ = 183°30' - 180° = 3°30' = N 3°30'E

Let x = PR and y = RQ

Latitude of R with respect to P = 193 - (x cos 39°30')

and, Latitude of R with respect to Q = 346 - (y cos 3°30')

∴ 193 - x cos 39°30' = 346 - y cos 3°30'

or, -x cos 39°30' + y cos 3°30' = 346 - 193 = 153

Similarly, x sin 39°30' + y sin 3°30' = 502 - 330 = 172

∴ x = 238 and y = 337

x cosθ = 238cos 39°30' = 183.7

x sinθ = 238 sin 39°30' = 151.4

∴ Co-ordinates of R =(193 - 183.7) = +9.3 and

(330 + 151.4) = +481.4

Co-ordinates of R = 9.3 N, 481.4E Ans.

Example 28. The following angular measurements are taken from two stations C and D, 600 metres apart, to two inaccessible points A and B.

ACB = 47°00', BCD = 58°0', BDA = 49°00', and ADC = 59°00'

Calculate the distance AB.

Answer : The figure indicates the stations C and D, 600m apart and the two inaccessible points A and B. The distance AB is to be found out.

In triangle ACD,

∠ADC = 59°

∠CAD = 180° - (47° + 58° + 49°) = 16°

By sine rule,

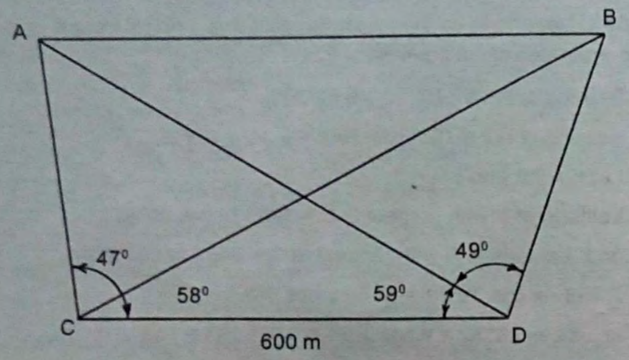
AC = $\frac{600\text{m} \times \text{Sin } 59^0}{\text{Sin } 16^0}$ = 1865.85 metres

In triangle BCD,

∠CBD = 180° - (58° + 59° + 49°) = 14°

∠CDB = 59° + 49° = 108°

$$BC = \frac{600 \text{ Sin } 108^0}{\text{Sin}14^0} = 2358.75 \text{ metres.}$$



Now, in triangle ACB,

$$AB = \sqrt{AC^2 + BC^2 - 2AC \times BC \times \text{Cos ACB}}$$

$$= \sqrt{1865.85^2 + 2358.75^2 - 2 \times 1865.85 \times 2358.75 \text{ Cos } 47^0}$$

$$= 1744 \text{ metres.}$$

Distance AB = 1744 metres. **Ans.**

Example 29. In the figure, given the length of AB = 273.6 metres. Calculate the length of line CD.

Answer : Let CD is assumed to be 100 metres.

In triangle ACD.

Angle ACD = $23^0 12' + 37^0 15' = 60^0 27'$

Angle CAD = $180^0 - (60^0 27' + 29^0 27') = 90^0 6'$

Applying sine rule :

$$\frac{AC}{\text{Sin } 29^0 27'} = \frac{CD}{\text{Sin } 90^0 6'} \qquad \therefore AC = \frac{100 \times \text{Sin } 29^0 27'}{\text{Sin } 90^0 06'} = 49.17\text{m.}$$

In triangle BCD = $180^0 - (37^0 15' + 29^0 27' + 16^0 42') = 96^0 36'$

$$\frac{BC}{\text{Sin } (29^0 27' + 16^0 42')} = \frac{CD}{\text{Sin } 96^0 36'}$$

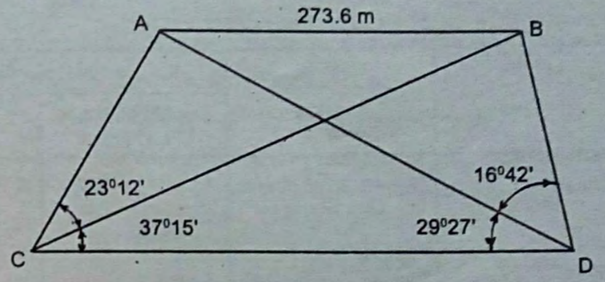
$$\therefore BC = \frac{100 \times \text{Sin } 46^0 09'}{\text{Sin } 96^0 36'} = 72.60\text{m.}$$

In triangle ABC, applying cosine rule.

$AB^2 = BC^2 + AC^2 - 2BC.AC.\text{Cos ACB}$

$= 72.60^2 + 49.17^2 - 2 \times 72.60 \times 49.17 \times \text{Cos } 23^0 12'$

$= 5270.8 + 2417.7 - 656.22 = 1126.3$



$AB = \sqrt{1126.3} = 35.56$ metres.

Thus when the length of AB is 35.56m. the length of CD = 100m.

Actually the length of AB = 273.60 metres.

Therefore, **the length of CD** = $273.60 \times \frac{100}{33.56} = 815.27$ metres.

Example 30. In order to determine the distance between two inaccessible points P and R, the following observations were made from two stations A and B, 400m apart -

PAR = 46^0, RAB = 56^0, ABP = 58^0 and

PBR = 48^0, Calculate the distance PR.

Answer : From the figure it is evident that the sides PA, AB and RP form a closed traverse.

Now in triangle PAB.

$\angle PAB = 46^0 + 56^0 = 102^0$, and

$\angle ABP = 58^0$

$\therefore \angle APB = 180^0 - (102^0 + 58^0)$

$= 20^0$

Again in triangle RAB,

$\angle ABR = 58^0 + 48^0 = 106^0$, and

$\angle RAB = 56^0$

$\therefore \angle ARB = 180^0 - (106^0 + 56^0)$

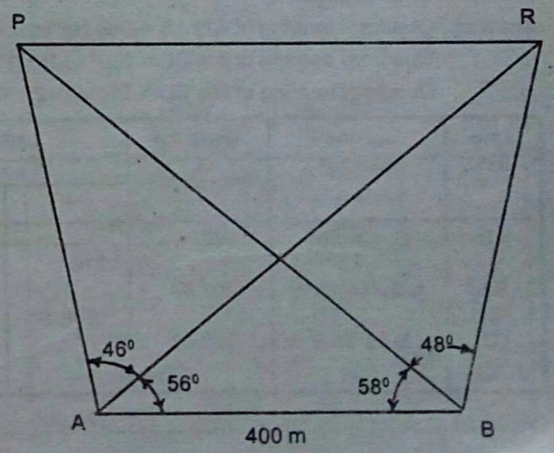
$= 18^0$

By sine rule, in triangle PAB,

$$PA = \frac{400 \times \text{Sin } 58^0}{\text{Sin } 20^0} = 991.8\text{m}$$

Similarly, from triangle ARB,

$$BR = \frac{400 \times \text{Sin } 56^0}{\text{Sin } 18^0} = 1073.1\text{m}$$



Let AP is the reference meridian, the bearing of AP is, therefore, zero.

Bearing of AP = 0° Due North.

Bearing of AB = S(180° - 102°)E = 78°E

Bearing of BR = (102° + 180° + 106°) - 360° = N28°E

Line Station	Length (metres) L	Reduced Bearing θ	Partial Co-ordinates	
			Latitude L Cosθ	Departure L Sinθ
P			0.00	0.00
PA	991.80	Due South		
A			-991.83	0.00
AB	400.00	S 78°E		
B			-83.16	+391.26
BR	1073.10	N 28°E		
R			+947.49	+503.78

ΣL = -127.47 ΣD = +895.04

The co-ordinates of R with respect to P as the origin = 127.47S. 895.04E,

Length of PR = $\sqrt{(\Sigma L)^2 + (\Sigma D)^2} = \sqrt{(127.47)^2 + (855.04)^2}$ = 904.07m.

Distance of PR = 904.07 metres Ans.

Example 31. The following bookings were obtained from a survey traverse book :-

Angle ABC = 118°15'18" and angle BCD = 108°40'42"

Distance AB = 276.00; BC = 207.87 and CD = 299.62m

Determine the co-ordinates of D relative to A, the length of AD and the angle BAD. The Line AB is in meridian.

Answer : Quadrant bearing of line AB = Due North.

Quadrant bearing of line BC = 180° - 118°15'18" = N 61°44'42"E

Quadrant bearing of line CD = 108°40'42" - 61°44'42" = S 46°56'E

Line Stn.	Quadrant Bearing θ	Distance L	Latitude L x Cosθ		Departure L x Sinθ	
			+	–	–	+
AB	Due North	276				
B			276	–	–	–
BC	N 61°44'42"E	207.87				
C			98.405	–	183.102	–
CD	S 45°56'E	299.62				
D			–	204.595	218.890	–
			374.405	-204.595	+401.992	

= +169.810

Total Co-ordinates of D relative to A = + 169.810 and + 401.992 **Ans.**

Bearing of line AD = N $\tan^{-1}\frac{401.992}{169.810}$ = N 67°06'E

The Line AB in meridian, therefore the angle BAD = 67°06' Ans.

Length of AD = Total departure of D x Cosec bearing

= 401.992 Cosec 67°06' = 436.385 metres **Ans.**

Example 32. The following are the notes of an underground traverse made from the centre of a shaft A to a point X. A roadway is to be driven from X dropping at 1 in 200 back to the point A but at a lower level

Line	Azimuth	Inclined Distance	Inclination
AB	95°30'	80.4m	Rising 1 in 50
BC	46°00'	40.2m	Level
CD	353°30'	102.6m	Dipping at 14°
DE	353°00'	87.0m	Dipping at 15°
EX	262°15'	98.4m	Rising 1 in 200.

Calculate the length and bearing of the roadway XA and determine how much the shaft must be deepened to reach the new roadway from X to A.

Answer : LENGTH AND BEARING OF A ROADWAY

Plan lengths of the lines are calculated as follows -

AB = $\frac{80.4 \times 50}{\sqrt{2501}}$ or, 80.4 x Cos 1°8'45" = 90.38m

BC = 40.2 x Cos 0° = 40.20m

CD = 102.6 x Cos 14° = 99.55m

DE = 87.0 x Cos 15° = 84.03m

EX = 98.4 x Cos 17'11" = 98.40m

Co-ordinates of X with respect to A :

Line	Bearing	Length	Partial Co-ordinates		Station
			Latitude	Departure	
AB	S 84°30'E	80.38m	- 7.70	+ 80.00	B
BC	N 46°00'E	40.20m	+ 27.92	+ 28.91	C
CD	N 6°30'W	99.55m	+ 98.91	- 11.27	D
DE	N 7°00'W	84.03m	+ 83.40	- 10.24	E
EX	S 82°15'W	98.40m	- 13.27	- 97.5	X
Total co-ordinates of X =			+ 189.26	- 10.10	

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Bearing of XA = $\tan^{-1} \frac{10.10}{189.26}$ = S 3°03'17"E

Length of XA = 189.26 x Sec 3°03'17" = 189.53m

Vertical difference between A and X -

Rise from A to B = $\frac{8.04}{50}$	= + 1.61m
Rise from B to C = Level	= + 0.00m
Fall from C to D = 102.6 x Sin 14°	= - 24.82m
Fall from D to E = 87 x Sin 15°	= - 22.52m
Rise from E to X = $\frac{98.4}{200}$	= + 0.49m
∴ Net fall from A to X	= - 45.24m

Let A' be the required point vertically below A. Then fall from X to A' in the lower level = $\frac{189.53}{200}$ = 0.95m

Therefore, the difference in level between A and A' = 46.19m.

Hence, to reach the new roadway the shaft have to be deepened by 46.19 metres.

Example 33. The co-ordinates of four stations relative to a common origin are respectively as follows:-

A	50.8 S	301.6 W
B	284.7 N	118.5 E
C	308.2 S	288.4 E
D	89.9 N	315.0 E

A and B are points in one roadway and C and D are points on another roadway. Calculate the co-ordinates of the point at which the two roadways will intersect when produced.

Answer : CO-ORDINATE CALCULATION

The figure shows the two roadways AB and CD. The two roadways intersect at point E when produced.

Length of BD = $\sqrt{(Lat_D - Lat_B)^2 + (Dep_D - Dep_B)^2}$

= $\sqrt{(89.9 - 284.7)^2 + (315 - 118.5)^2}$ = 276.69

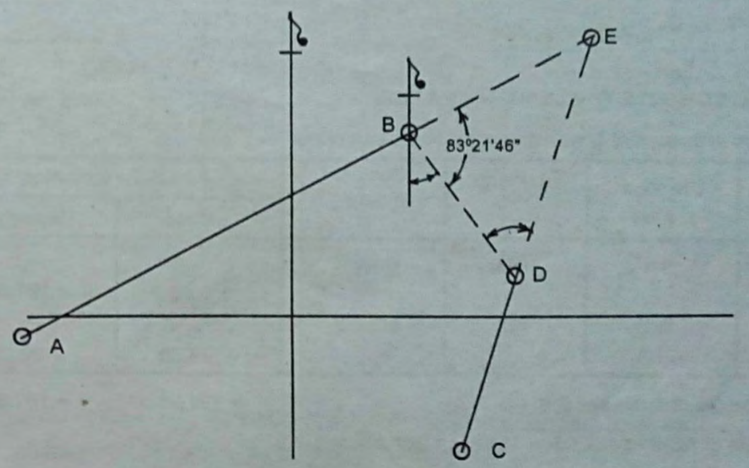
Bearing of BD = $Tan^{-1} \frac{(Dep_D - Dep_B)}{(Lat_D - Lat_B)}$ SE

= $\tan^{-1} \frac{315 - 118.5}{89.9 - 284.7}$ SE

= S 45°14'56" E

Bearing of AB = $\tan^{-1} \frac{118.5 - (-301.6)}{284.7 - (-50.8)}$ NE

= N 51°23'18" E



Bearing of CD = $\tan^{-1} \frac{315 - 288.4}{89.9 - (308.2)}$ NE

= N 3°49'22" E

Angle EBD = 180° - (45°14'56" + 51°23'18") = 83°21'46"

Angle BDE = 45°14'56" + 3°49'22" = 49°04'18"

Angle BED = 180° - (83°21'46" + 49°04'18") = 47°33'56"

$\frac{BE}{\text{Sin BDE}} = \frac{BD}{\text{Sin BED}}$

BE = 276.69 x $\frac{\text{Sin } 49°04'18"}{\text{Sin } 47°33'56"}$ = 283.24

Partial Latitude of E = 283.24 x Cos 51°23'18" = 176.75 (+)

Partial Departure of E = 283.24 x Sin 51°23'18" = 221.32 (+)

Hence the **total co-ordinates of the point E** are –

Latitude = + 284.7 + 176.75 = 461.45

Daparture = + 118.5 + 221.32 = 339.82

Example 34. For the following traverse compute the length CD so that A, D and E may lie in one straight line. Distances are in metres.

Line	Distance	Bearing
AB	102	85°
BC	151	32°
CD	-	350°
DE	186	18°

Answer : TO COMPUTE THE LENGTH OF A LINE

The co-ordinates of C with respect to A are calculated -

Line Station	Distance L mts.	Azimuth	Quadrant Bearing θ	Partial Co-ordinates	
				Latitude	Departure
AB	102	85°	N 85°E		
B				+ 8.89	+ 101.61
BC	151	32°	N 32°E		
C				+ 128.06	+ 80.02
Total Co-ordinates of C =				+ 136.95	+ 181.63

Length of AC = $\sqrt{136.95^2 + 181.63^2}$ = 227.47 m

Bearing of AC = $\tan^{-1} N \frac{181.63}{136.95} E$ = N 52°59'01"E

In triangle ADC, Bearing of AD = Bearing of DE = 18°

∴ ∠DAC = Bearing of AC - Bearing of AD = 34°58'01"

∠ADC = Bearing of DA - Bearing of DC = (18° + 180°) - (350° - 180°)

= 198° - 170° = 28°

∠ACD = 180° - (34°59'01" + 28°) = 117°00'59"

Now, in triangle ADC,

Length of CD = AC x $\frac{\text{Sin A}}{\text{Sin D}}$ = 227.47 x $\frac{\text{Sin } 34°59'01"}{\text{Sin } 28°}$ = 227.80 metres.

Example 35. From a point C it is required to set out a line CD parallel to given line AB, so that the angle ABD is a right angle. C and D are invisible from A and B and traversing is made as follows :

Line	Length	Bearing
BA	-	360°
BE	77.50 m	290°57'
EF	92.00 m	352°06'
FC	59.00 m	263°27'

Calculate the length and bearing of CD.

Answer : TO SET OUT A LINE PARALLEL TO A GIVEN LINE

The co-ordinates of C with respect to B are calculated as follows -

LineStn.	Azimuth	Quadrant Bearing θ	Length L	Partial Co-ordinates	
				Latitude L x Cosθ	Departure L x Sinθ
BE	290°57'	N 69°03'W	77.50		
E				+ 27.71	- 72.33
EF	352°06'	N 7°54'W	92.00		
F				+ 91.13	- 12.64
FC	263°27'	S 83°27'W	59.00		
C				- 6.73	- 58.61
Total Co-ordinates of C =				+ 112.11	- 143.63

Length of BC = $\sqrt{(112.11)^2 + (-143.68)^2}$ = 162.20 m.

Bearing of BC = $\tan^{-1} \frac{-143.63}{112.11}$ = N 51°01'35"W = 307°56'25"

Bearing of BD = 270°. Since CD is parallel to AB and BD is perpendicular to CD,

∠DBA = 90°.

∠DBC = Azimuth of BC - Azimuth of BD = 370°58'25" - 270° = 37°58'25"

The Length of CD = BC x Sin CBD = 182.20 x Sin 37°58'25" = 112.11 m

Bearing of CD = Bearing of AB = 180° or **Due South.**

Example 36. The co-ordinates of three stations A, B and C are given :

Station	Latitude (Metres)	Departure (Metres)
A	+ 10,211.3	+ 5,269.8
B	+ 4,326.8	- 54.6
C	– 1,020.4	+ 2,081.7

A line is set out from C on a bearing 265°15'03". Determine the co-ordinates of the point of intersection of the lines AB and CD.

Answer : CO-ORDINATE PROBLEM.

Let X is the point of intersection of the lines AB and CD.

Quadrant bearing AB = tanα

= $\frac{\text{Diff. of Deps.}}{\text{Diff. of Lats.}}$

= $\frac{5269.8 - (-54.6)}{10211.3 - 4326.8}$

= 42°08'22" SW

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Quadrant bearing AC

$$= \frac{5269.8 - 2081.7}{10211.3 - (-1020.4)}$$

$= 15^0 50'47"$ SW

Length of AC = (10211.3 + 1020.4) Sec. $15^0 50'47"$

= 11675.1 m.

Angle at A = $42^0 08'22" - 15^0 50'47"$

= $26^0 17'35"$

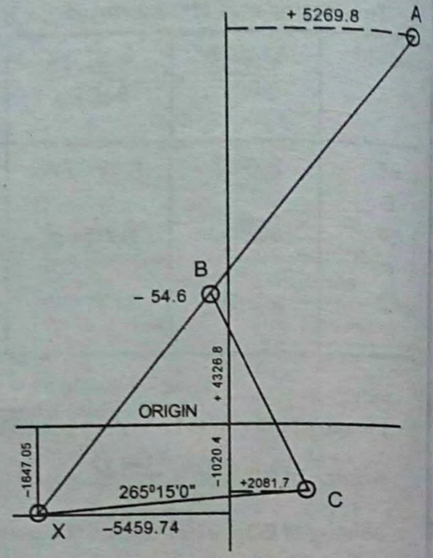
Angle at X = $(265^0 15'00" - 110^0 00'00") - 42^0 08'22"$

= $43^0 06'38"$

By the Sine Rule = $\frac{AC}{\text{Sin X}} = \frac{CX}{\text{Sin A}}$

$$\frac{11675.1}{\text{Sin } 43^0 06'38"} = \frac{CX}{\text{Sin } 26^0 17'35"}$$

Therefore, CX = 7567.43 metres.



Quadrant bearing of line CX = $265^0 15'00" - 180^0$

= $85^0 15'00"$ SW

Partial latitude of X in relation to C :

7567.43 Cos $85^0 15'$ = 626.65 m (South)

Partial dep. of X in relation to C :

7567.43 Sin $85^0 15'$ = 7541.44 m (West)

Therefore, Total latitude of X =

Total lat of C + Partial lat of X

= –1020.40 + (–626.65) = - 1647.05 m

Similarly, Total departure of X =

- 7541.44 + 2081.70 = - 5459.74 m.

Co-ordinates of X : 11647.05 (S), 5459.74 (W).

Example 37. The following survey notes refer to a boundary traverse and station A and E a situated on the boundary.

Line	Bearing	Hor. Length
AB	N $83^0 15'$E	253.2 m
BC	S $40^0 30'$E	426.4 m
CD	N $36^0 13'$E	543.8 m
DE	S $23^0 54'$E	1260.2 m

It is proposed to replace the boundary ABCDE by boundary AXE where AX is a straight l

and X is situated on the line DE. Calculate the distance EX which will give equalisation of areas on each side of the boundary.

Answer : Equalisation of Areas :

Stn.	Partial Co-ordinates		Total Co-ordinates	
	Latitude	Departure	Latitude	Departure
A			0	0
B	N 29.83	E 251.44	N 29.83	E 251.44
C	S 293.51	E 309.30	S 263.68	E 560.74
D	N 438.73	E 321.30	N 175.05	E 882.04
E	S 1152.15	E 510.56	S 977.10	E 1392.60

These co-ordinates are roughly plotted. The area of the figure ABCDE is now calculated by applying the following rule : Multiply the total latitude of each station in turn by the algebraic sum of its partial departure and that of the station next following. Then the area is half the algebraic sum of all the products thus obtained.

The necessary figures are set out as follows :

Station	Total Lat	Partial Dep	Sum of Adjacent Deps		Double Areas +	Double Areas -
A	0	-1392.60	–1141.16	(A + B)	0	-
B	+ 29.63	+251.44	+560.74	(B + C)	16,615	-
C	–263.68	+309.30	+630.60	(C + D)	-	166.277
D	+175.05	+321.30	+831.86	(D + E)	145,617	-
E	-977.10	+510.56	-882.04	(E + A)	861,841	-
					1,024,073	166.277
					- 166,277	
				Double Area =	857,796	

Therefore area of figure ABCDEA = 857796 ÷ 2 = 428,900 square metres

Bearing of line AE = $\tan^{-1} \frac{1392.60}{977.10}$

= S $54^0 57'$E

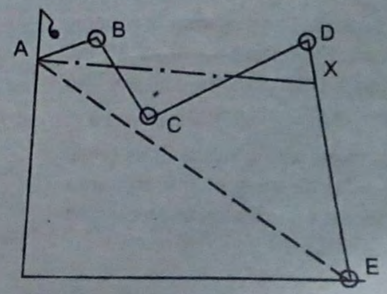
Length of AE = 1392.60 x Cosec $54^0 57'$

= 1701.1 metres.

Bearing AE – Bearing DE = Angle AED

Angle AED = S $54^0 57'$E – S $23^0 54'$E

= $31^0 03'$



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To find the distance EX : The condition for EX to straighten the boundary is that the area of triangle AEX must be equal in area to the figure ABCDEA. In the triangle AEX, we have found that AE = 1701.1 m.

Angle AEX = Angle AED = 31°03'

Therefore area of triangle AEX = $\frac{AB \times EX \times \text{Sin AEX}}{2}$

$$428900 = \frac{1701.1 \times EX \times \text{Sine } 31°03'}{2}$$

$$EX = \frac{428900 \times 2}{1701.1 \text{ Sine } 31°03'}$$

Therefore, the **distance EP** = 977.7 metres

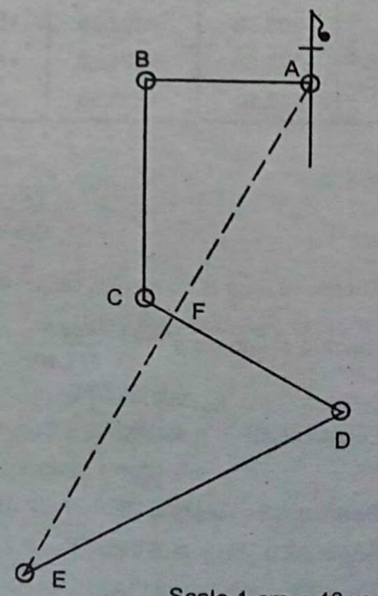
Example 38. The following are the notes of an underground traverse made with a theodolite in part of a main airway in level seam :

Line	Azimuth	Distance
AB	271°30'	120.00m
BC	184°48'	165.00m
CD	120°00'	147.00m
DE	238°30'	234.30m

It is proposed to improve the ventilation by driving a new road way in direct line between the points A and E, and in order to complete the work with the maximum of speed to drive simultaneonsly on four separate faces. Calculate the position of point F on the line CD at which two faces are to be set out and state the quadrant bearing of FE.

Ans. The traverse is roughly plotted. From the bearing of AE the triangle FDE can be solved, the distance DF calculated.

Now taking station A as origin the co-ordinates of the traverse stations are calculated and tabulated below :



Scale 1 cm = 40 m

Line Stn.	Distance mts.	Azimuth	Quadrant Bearing θ	Partial Latitude (L cosθ)	Partial Departure (L sinθ)	Total Co-ordinates	
						Latitude	Departure
A						0	0
AB	120	271°30'	N 88°30'W				
B				+3.1412	–119.95	+3.1412	–113.95
BC	165	184°48'	S 4°48'W				
C				–164.40	–13.804	–261.26	–133.754
CD	147	120°00'	S 60° E				
D				–73.485	+127.30	–234.745	–006.454
DE	234.3	238°30'	S 58°30'W				
E				–122.40	–199.76	–357.145	–206.214

Bearing of AE = $S\left(\tan^{-1}\frac{206.214}{367.145}\right)$ W = S 30°W

Bearing of FE = Bearing of AE = S 30°W

In triangle FDE, ∠DEF = Azimuth of FE - Azimuth of FD

= (180° + 30°) - 120° = 90°.

∠EDF = Azimuth of DF - Azimuth of DE = (120° + 180°) - 238°30' = 61°30'

∠FED = AZimuth of ED - Azimuth of EF = (238°30' - 180°) -30° = 28°30'

Now in the right angle triangle FDE, ED - 234.3 m and FED = 28°30'

Therefore FD = ED sin FED = 234.3 x sin 28°30' = 111.80 m.

The position of F is such that the distance FD = 111.80 metres.

Bearing of FE = S 30°W

Example 39. The observed bearings and horizontal lengths of a closed traverse are as follows :

AB	N 60°30'E	230 m
BC	N 88°15'E	400 m
CD	S 50°20'W	500 m
DA	N 48°06'W	292 m

The total co-ordinates of station A are E 3652.19m and N 4321.74m. Calculate the error of closure of the traverse and adjust it by the 'Bowditch' method. State the corrected co-ordinates of station D.

Answer : TRAVERSE CORRECTION

Co-ordinates of the stations are calculated to ascertain the closing error of the traverse :

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Station	Line	Quadrant Bearing	Length	Partial Co-ordinates		Total Co-ordinates	
				Lat	Dep	Lat	Dep
A						+4321.74	+3652.19
B	AB	N 60°30'E	230m	+113.26	+200.18	+4435.00	+3852.37
C	BC	N 88°15'E	400m	+ 12.22	+399.81	+4447.22	+4252.18
D	CD	S 50°20'W	500m	-319.13	-384.89	+4128.09	+3867.29
A	DA	N 48°06'W	292m	+195.91	-217.34	+4323.10	+3649.95
						4323.10	3649.95
	Sum =			+ 1.36	- 2.24	4321.74	3552.19
						+ 1.36	- 2.24

Error of closure : Lat = + 1.36 m, and Dep = - 2.24 m.

Bowditch's rule assumes that errors at each station are proportional to the length of the preceding leg, i.e. greater errors will occur in measuring lengths than in measuring angles.

Total distance covered by survey = 230 + 400 + 500 + 291 = 1422 metres.

Latitude correction factor = 2.36 ÷ 1422

Departure correction factor = 2.24 ÷ 1422

Therefore by multiplying algebraically the inter-station distances by these two correction factors, subtracting them from the partial latitudes (since the error is positive), and adding the result to the partial departures (because the error is negative in this case) the appropriate adjustments we made.

Co-ordinates sheet after Adjustment.

Stn.	Line	Latitude Correction Factor 'N' = 1.36 ÷ 1422 x interstation	Departure correction Factor 'M' = 2.24 ÷ 1422 x interstation	Adjusted Total Co-ordinates	
				Total Co-ordinate + (Partial Co-ordinate ± correction)	
				Latitude	Departure
A		- Subtract	+ Add	+4321.740	+3652.190
B	AB	N x 230 = 0.220	M x 230 = 00.362	+4434.780	+3852.732
C	BC	N x 400 = 0.382	M x 400 = 0.630	+4446.618	+4353.172
D	CD	N x 500 = 0.478	M x 500 = 0.787	+4127.010	+3869.069
E	DA	N x 292 = 0.280	M x 292 = 0.461	+4321.740	+3652.190
	Sum Total	+ 1.360	2.240	Adjusted	Adjusted

Corrected co-ordinates of station D are :

Latitude = N 4127.010; Departure = E 3869.069

Example 40. The following theodolite traverse was carried out between the faces of two advancing roadways, BA and FG which are to be driven until they meet. Calculate the distance still to be driven in each roadway.

Line	Azimuth	Distance
AB	267°55'	45m
BC	355°01'	105m
CD	1°41'	94.5m
DE	0°53'	150.9m
EF	86°01'	318m
FG	203°55'	126m

Answer : MEETING POINT OF TWO ADVANCING FACES

The Co-ordinates of each station are calculated and tabulated :

Line Station	Azimuth	Quadrant Bearing	Distance (metres)	Partial Co-ordinates	
				Latitude	Departure
AB	267°55'	S 87°55'W	45.0		
B				– 1.636	– 44.970
BC	355°01'	N 4°59'W	105.0		
C				+ 104.603	– 9.121
CD	1°41'	N 1°41'E	94.5		
D				+ 94.460	+ 2.776
DE	0°53'	N 0°53'E	150.9		
E				+ 150.882	+ 2.326
EF	86°01'	N 86°01'E	318.0		
F				+ 22.090	+ 317.232
FG	202°55'	S 23°55'W	126.0		
G				– 115.181	– 51.081
		Total Co-ordinates of G =		+ 255.218	+ 217.162

The tangent of bearing AG = $\frac{217.162}{255.218}$ = N 40°23'40"E

Length of AG = Latitude x Secant 40°23'40"

= 255.218 x 1.3130 = 335.107 metres.

Let K is the point of intersection of the two anvancing faces BA and FG.

In triangle AKG :

Angle GAK = 87°55' – 40°23'40" = 47°31'20"

Angle KGA = 40°23'40" – 23°55' = 16°28'40"

Angle AKG = 180° – (47°31'20" + 16°28'40") = 116°00'

Applying the sine rule :

$$\frac{AK}{\text{Sine } 16°28'40"} = \frac{335.107}{\text{Sine } 116°} \qquad \therefore AK = 105.754\text{m}$$

$$\frac{GK}{\text{Sine } 47°31'20"} = \frac{335.107}{\text{Sine } 116°} \qquad \therefore GK = 274.985\text{m}$$

Therefore, Distance BA has to be driven = 105.754 metres.

Distance FG has to be driven = 274.985 metres.

Example 41. The following are the total co-ordinates of a five sided polygon. Calculate its area.

Station	Total Latitude (m)	Total Departure (m)	Remarks
A	0	0	
B	+ 564.73	+ 376.41	
C	– 185.67	+ 1257.63	
D	– 969.74	+ 604.05	
E	– 1142.18	+ 548.33	

Answer : AREA OF A POLYGON

One rule to find the area of a closed polygon from the co-ordinates of the corner stations of the enclosure is as follows : To multiply the sum of adjacent total latitudes by the difference of adjacent total departures, and to divide the algebraic sum of the products by two to get the area of polygon. The calculations are given as follows :

Stn.	Total Lat (m)	Total Dep. (m)	Sum of adj. Total Lats.	Difference of adj. Deps.	Double Areas (Sq. m)	
					+	–
A	0	0	+ 564.73	– 376.41		212570.02
B	+ 564.73	+ 376.41	+ 379.06	– 881.22		334035.25
C	– 185.67	+1257.63	–1155.41	+ 653.58		755152.87
D	– 969.74	+ 604.05	–2111.92	+1152.38		2433734.40
E	–1142.18	– 548.33	–1142.18	– 548.33		
A	0	0			626291.56	
					626291.56	3735492.54
						615291.56
					Double area =	3109201

Hence area of polygon = 3109201 ÷ 2 = 1554600.5 sq.m.

Example 42. The following are the notes of a traverse survey from station A to station F :

Line	Length (m)	Azimuth Bearing	Inclination
AB	50.00	330°00'	Horizontal
BC	32.40	45°10'	Horizontal
CD	100.00	90°00'	+ 5°00'
DE	47.60	132°04'	- 8°30'
EF	73.00	180°00'	Horizontal

Co-ordinates of station A are latitude 1000.00m and departure 1000.00m. Calculate the total latitude and total departure of each station and determine the length and bearing of line FA.

Answer : CO-ORDINATE CALCULATIONS

The inclined lengths are reduced to horizontal lengths :

CD 100m rising 5°00' = 100 x cos5° =99.62m

DE 47.60m dipping 8°30' = 47.60 x cos 8°30' = 47.08m

The co-ordinates of stations are calculated as follows :

Line Stn.	Hor. dist. m	Q.B. θ	Partial Co-ordinates		Total Co-ordinates	
			Latitude	Departure	Latitude	Departure
A					1000.00	1000.00
AB	50.00	N 30°W				
B			+ 43.30	– 25.00	1043.30	975.00
BC	32.40	N 45°10'E				
C			+ 22.84	+ 22.98	1066.14	997.98
CD	99.62	Due East				
D			+ 99.62	00.00	1165.76	997.98
DE	47.08	S 47°56'E				
E			- 31.54	+ 34.95	1134.22	1032.93
EF	73.00	Due South				
F			0.00	– 73.00	1134.22	959.93

Length of FA = $\sqrt{(1134.22 - 1000)^2 + (959.93 - 1000)^2}$

= **140.07 metres**

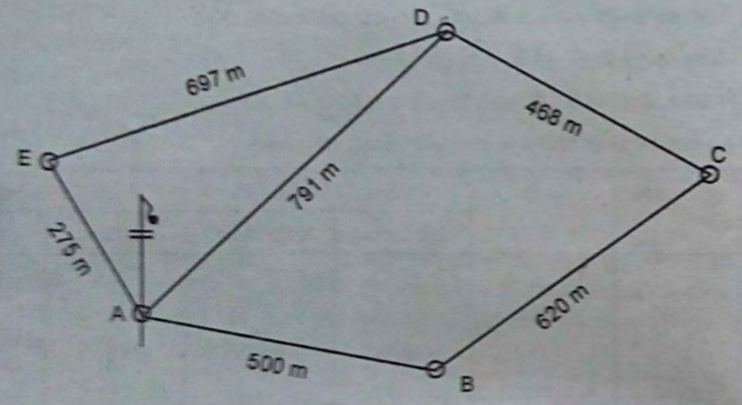
Bearing of FA = $S\left(\tan^{-1}\frac{40.07}{134.22}\right) E$ = **S 16°27'20"E**

Example 43. Following are the observations of a closed traverse. The lengths DE and EA could not be measured due to obstacles in the route :

Side	Length (m)	Azimuth
AB	500	98°30'
BC	620	30°20'
CD	468	298°30'
DE	?	230°00'
EA	?	150°10'

Find out the missing lengths DE and AE, by plotting the traverse to a suitable scale. Check your results by calculating the missing lengths from the data given above.

Answer : CO-ORDINATE CALCULATIONS FOR MISSING LENGTHS



The traverse is plotted to a scale 1 cm = 60 metres. The line DE and AE intersect at E. The lengths AE and DE measured. Measured length of DE = 697 m and AE = 275 m.

Calculations : The azimuths of AE, BC and CD are converted to quadrant bearings and then the co-ordinates of B,C and D are calculated as shown in the table :

Line / Stn.	Length	Azimuth	Q.B.	Partial Co-ordinates	
				Latitude	Departure
A				0	0
AB	500	98°30'	S 81°30' E		
B				– 73.90	+ 494.51
BC	620	30°20'	N 30°20' E		
C				+ 535.12	+ 313.12
CD	468	298°30'	N 61°30' W		
D				+ 223.31	– 411.29
	Total Co-ordinates of D =			+ 684.53	+ 396.35

Length of AD = $\sqrt{Lat^2 + Dep^2} = \sqrt{684.53^2 + 396.35^2}$

= 790.996 m.

Bearing of AD = $\text{Tan}^{-1}\left(\frac{396.35}{684.53}\right)$ NE = N 30°04'E

Bearing of DE = S 50°W and Bearing of DA = S 30°04'W

Hence $\angle ADE$ = S 50°W - S 30°04'W = 19°56'

Bearing of AE = N 29°50'W and Bearing of AD = N 30°04'E

$\therefore \angle EAD$ = 29°50' + 30°04' = 59°54'

Form Sine Rule : In triangle AED -

$$\frac{AE}{\text{Sin ADE}} = \frac{DE}{\text{Sin EAD}} = \frac{AD}{\text{Sin AED}}$$

$$\therefore AE = \frac{AD \times \text{Sin ADE}}{\text{Sin AED}} = \frac{790.996 \times \text{Sin } 19°56'}{\text{Sin } [180° - (19°56' + 59°54')]}$$

= 273.95 m.

$$\text{and } DE = \frac{AD \times \text{Sin EAD}}{\text{Sin AED}} = \frac{790.996 \times \text{Sin } 59°54'}{\text{Sin } 100°10'}$$

= 695.27 m.

Example 44. The co-ordidates in metres (reduced to a local origin) of three survey stations of the National Grid are as follows :

	Easting	Northing
X	1011.10	3104.35
Y	3898.56	5838.71
Z	6833.74	4805.26

Observations taken from a station A to the south of station Y have given the undernoted informations :

Angle XAY : 53° - 14' - 55"

Angle YAZ : 36° - 12' - 06"

Calculate the co-ordinates of station A in relation to the local origin and the bearing of the line AY.

Answer : Figure shows the relative positions of the survey stations and the station A.

Easting of Y relative to X = 3898.56 - 1011.10 = 2887.46 m

Northing of Y relative to X = 5838.71 - 3104.35 = 2734.36 m

Bearing of line XY = N $\tan^{-1}\frac{2887.46}{2734.36}$ E = N 46°33'36"E

Length of XY = $\sqrt{(2887.46)^2 + (2734.36)^2}$ = 3976.70 m

Easting of Z relative to Y = 2935.18 m

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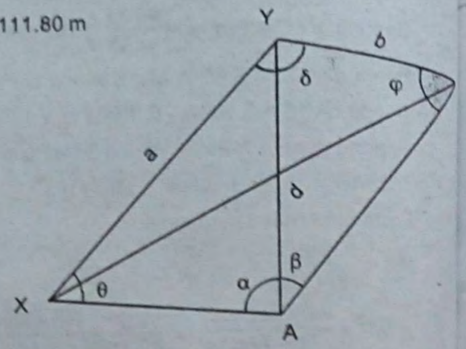
Southing of Z relative to Y = 1033.45 m

Bearing of line YZ = S $\tan^{-1}\frac{2235.18}{1033.45}$ E = S 70°36'10"E

Length of YZ = $\sqrt{(2935.18)^2 + (1033.45)^2}$ = 3111.80 m

Angle ZYX = (46°33'36" + 70°36'10")

= 117°09'46"



Now let XY = a, ZY = b,

Angle ZYX = δ,

In triangle XAY :

$\frac{a}{\sin\alpha} = \frac{d}{\sin\theta}$; or $d = \frac{a\sin\theta}{\sin\alpha}$

In triangle YAZ ;

$\frac{b}{\sin\beta} = \frac{d}{\sin\phi}$; or $d = \frac{b\sin\phi}{\sin\beta}$

Then $\frac{a\sin\theta}{\sin\alpha} = \frac{b\sin\phi}{\sin\beta}$ whence $\frac{\sin\phi}{\sin\theta} = \frac{a\sin\beta}{b\sin\alpha}$

Let $\theta + \phi = \Sigma = 360 - (\alpha + \beta + \delta)$ and $\phi = (\Sigma - \theta)$

Therefore $\frac{\sin\phi}{\sin\theta} = \frac{\sin(\Sigma - \theta)}{\sin\theta} = \frac{\sin\Sigma\cos\theta - \cos\Sigma\sin\theta}{\sin\theta}$

$= \frac{a\sin\beta}{b\sin\alpha}$ and $\sin\Sigma\cot\theta - \cot\Sigma = \frac{a\sin\beta}{b\sin\alpha}$

From this we get $\sin\Sigma\cot\theta = \frac{a\sin\beta}{b\sin\alpha} + \cos\Sigma$

If we substitute the known values into the equation,

$\cot\theta = \frac{3967.7\ \sin 36°12'06"}{3111.18\ \sin 53°14'55"\ \sin 153°23'13"} + \cot 153°23'13"$

Therefore, $\cot\theta = 2.102825 - 1.995826 = 0.160999$

and $\theta = 83°53'27"$

In triangle XAY :

Angle Y = 180° – (83°53'27" + 53°14'56") = 42°51'38"

$\frac{a}{\sin\alpha} = \frac{AX}{\sin Y}$ and $AX = \frac{a\sin\gamma}{\sin\alpha} = \frac{3967.7\ \sin 42°51'38"}{\sin 53°14'25"}$

Therefore AX = 3376.04 metres.

Bearing of line XA = Bearing of XY + θ

= 46°33'36" + 83°53'27" = 130°27'03"

= S 49°33'57" E

Easting of A relative to X = 3376.04 Sin 49°33'57" = 2569.65

Therefore, **Easting of A relative to local origin**

= (1011.10 + 2569.65) = **3580.75 metres.**

Southing of A relative to X = 3376.04 Cos 49°33'57" = 2189.57

Therefore, **Southing of A relative to local origin**

= (3104.35 - 2189.57) = **914.78 metres**

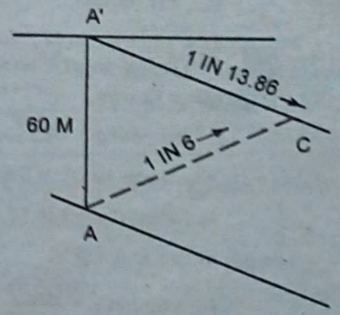
Bearing of line AY = N (46°33'26" - 42°51'38") E

= N 3°41'48" E.

Example 45. Two parallel levels 60m (inclined distance) apart run due East and West in a seam which dips due north at 1 in 12 . At a point A in the lower level a cross-measure drift rising at 1 in 6 and bearing N 39°E is driven to intersect another seam, situated 60m vertically above the seam first mentioned, at a point C. From a point B in the upper level due south from A another cross-measure drift rising at 1 in 3 and bearing N 30°E is also driven to intersect the upper seam at a point D. Calculate the length and azimuth of the line CD. It can be assumed that the top seam is also having the same dip and direction as that of the bottom seam.

Answer : The problem is roughly plotted.

Plan length of AB = $\frac{60 \times 12}{\sqrt{145}}$ = 59.793 m



Apparent dip of the upper seam towards N 30°E :

Tan app, dip = Tan full dip x cos30°

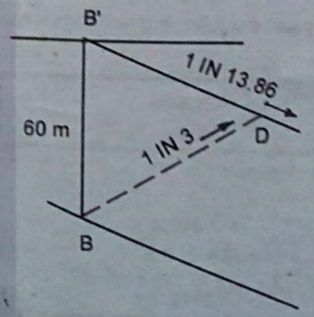
$= \frac{1}{12} \times 0.866$

= 0.0721687

∴ Apparent dip = 1 in 13.86

Rate of approach for the drift AB rising 1 in 6

$= \frac{1}{13.86} + \frac{1}{6} = \frac{19.86}{83.16}$



Plan length of drift AC = $60 \times \frac{83.16}{19.86}$

= 251.239 m

Rate of approach for the drift BD rising 1 in 3

$= \frac{1}{13.86} + \frac{1}{3} = \frac{16.86}{41.58}$

Plan length of drift BD = $60 \times \frac{41.58}{16.86}$

= 147.97 m

To calculate the length and bearing of the closing line CD :

Line	Horizontal Length	Quadrant Bearing	Latitude +	Latitude −	Departure +	Departure −
CA	281.239	S 30°W	–	217.580	–	125.619
AB	59.793	Due South	–	59.793	–	–
BD	147.970	N 30°E	128.146	–	73.985	–
			128.146	277.373	73.985	125.619
				128.146		73.985
				149.227		51.634

$$\text{Bearing of DC} = \text{Tan}^{-1}\frac{-51.634}{-149.227} = \text{N } 19°05'\text{E}$$

Bearing of CD = S 19°05'W.

Horizontal Distance of DC

$$= \sqrt{(-149.227)^2 + (-51.634)^2}$$

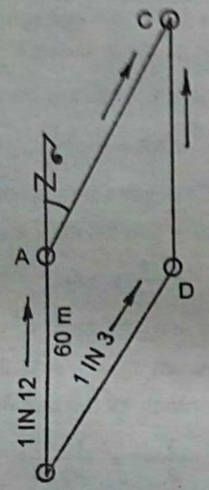
= 157.907 metres.

Full dip of the seam is 1 in 12 towards North, hence the apparent dip towards DC having bearing N 19°06'E will be as follows :

$$\text{Tan apparent dip} = \text{Tan full dip} \times \cos\theta$$

$$= \frac{1}{12} \times \cos 19°05' = 0.0787536$$

∴ Apparent dip = 1 in 12.7



Length of the line CD

$$= \text{Plan length} \times \frac{\sqrt{(12.7)^2 + 1}}{12.7}$$

= 157.907 x 1.0031 = **158.396 m.**

Example 46. A traverse was run along the boundary of a colliery and the following readings were obtained :

Line	Distance In Metres	Azimuth Bearing	Remarks
AB	1432	270°	
BC	1766	344°	
CD	2167	97°30'	EA is the
DE	900	135°	Closing line

It is proposed to extend the lines BA and DE to meet at F and to acquire the land defined by area AEF. Calculate the area AEF.

Answer : AREA OF A TRIANGULAR LAND

Q.B. of AB = 90° Due West

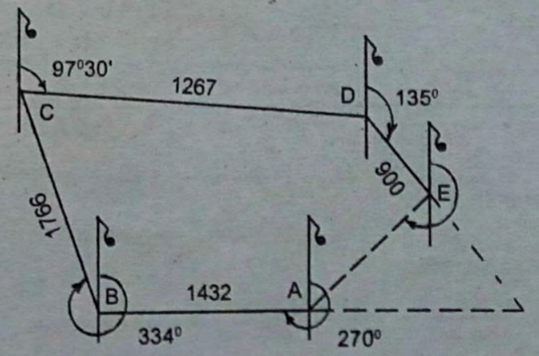
Q.B. of BC = N 16° W

Q.B. of CD = S 82°30' E

Q.B. of DE = S 45° E

Station	Line	Q. B.	Distance in m	PARTIAL CO-ORDINATES Latitude N	Latitude S	Departure E	Departure W
A							
	AB	90° due W	1432				
B							1432.00
	BC	N 16° W	1766				
C				1697.59			486.77
	CD	S 82°30' E	2167				
D					282.85	2148.46	
	DE	S 45° E	900				
E					636.40	636.40	
				1697.59	919.25	2784.86	1918.77
	EA				778.34		866.09
			Add	1697.59	1697.59	2784.86	2784.86

Since the latitude and departure of the closing line EA are both negative, the line lies in the 3rd quadrant.



Let the Q.B. of the closing line = θ

$$\text{Then, } \tan\theta = \frac{\text{Departure}}{\text{Latitude}} = \frac{866.09}{778.34} = 1.1127$$

$\therefore \theta = 48.03^0$

Length of the closing line EA

= Departure cosecθ

= 866.09 cosec 48°3'

= 1164.57 metres.

For solution of the △AEF

W.C.B. of EA = 48°3' + 180°0' = 228°3'

W.C.B. of EF = W.C.B. of DE = 135°0'

$\therefore \angle E$ = 93°3'

W.C.B. of AF = 90°0'

W.C.B. of AE = 48°3'

$\therefore \angle A$ = 41°57'

W.C.B. of FE = 135° + 180° = 315°0'

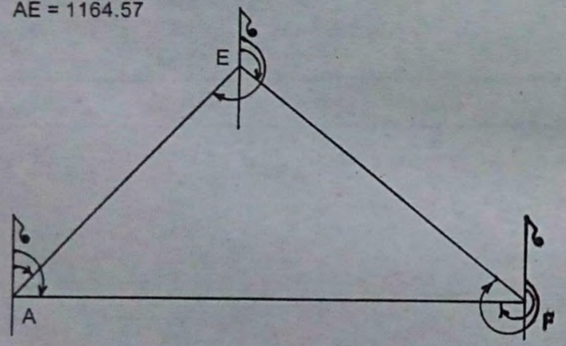
W.C.B. of FA = W.C.B. of AB = 270°0'

$\therefore \angle F$ = 45°0'

$\angle A + \angle E + \angle F = 41^0 57' + 93^0 3' + 45^0 0' = 180^0 0'$

In the △AEF,

AE = 1164.57



To find the length of EF and FA.

$$\frac{AE}{\text{Sin } 45^0} = \frac{AF}{\text{Sin } 93^0 3'}$$

$$\text{or } AF = \frac{AE \text{ Sin } 93^0 3'}{\text{Sin } 45^0} = \frac{1164.57 \times 0.998}{0.7071} = 1643.68 \text{ metres.}$$

Again, $\frac{AE}{\text{Sin } 45^0} = \frac{EF}{\text{Sin } 41^0 57'}$

$$\text{or } EF = \frac{AE \text{ Sin } 41^0 57'}{\text{Sin } 45^0} = \frac{1164.57 \times 0.6684}{0.7071} = 1100.83 \text{ metres.}$$

Thus,

AE = 1164.57 metres.

EF = 1100.83 metres.

FA = 1643.68 metres.

$$S = \frac{f + a + e}{2} = \frac{1164.57 + 1100.83 + 1643.68}{2} = 1954.54 \text{ metres.}$$

(S - f) = 1954.54 - 1164.57 = 789.97

(S - a) = 1954.54 - 1100.83 = 853.71

(S - e) = 1954.54 - 1643.68 = 310.86

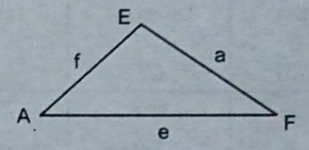
Area of the △AEF

$= \sqrt{S\,(S - f)\,(S - a)\,(S - e)}$

$= \sqrt{1954.54 \times 789.97 \times 853.71 \times 310.86}$

= 44.21x 28.10 x 29.29 x 17.63

= 1242.3 x 516.38 = 641 499 Sq. metres = 64.15 Hectores



Example 47. The co-ordinates in metres of the corners of the figure ABCDEF are givenbelow :

	Latitude	Departure
A	207 N	331 W
B	1309 N	98 W
C	1508 N	567 E
D	504 N	777 E
E	118 S	498 E
F	370 S	86 W

Form A a line is to be set out to divide the figure into two equal areas. Calculate the bearing of the line and also the co-ordinates of the point where the line cuts the side of this figure.

Answer : CO-ORDINATE CALCULATIONS

Area of ABCDEFA

Stn.	Total Lat.	Total Dep.	Difference in Dep of Adj. Stns	Double Area (Cols 2 x 4)	
				+	–
1	2	3	4		
A	207	–331	–12		2484
B	1309	– 98	898	1175482	
C	1508	567	875	1319500	
D	504	777	–69		34776
E	–118	498	–863	101834	
F	–370	–86	–820	306730	
				2903546	37260
				37260	
			Algebraic Sum =	2866286	

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$\therefore$ Area = $\frac{1}{2}$ x Algebraic Sum = $\frac{2866286}{2}$ = 1433143 Sq. m.

Let the line from A cuts the side of the figure at X = dividing the figure into two equal areas :

Area of ABCXA = $\frac{1}{2}$ of the Area of 1433143 Sq.m.

= 716572 Sq.m.

Area of ABCDA

Stn. (1)	Total Lat. (2)	Total Dep. (3)	Difference in Dep. to Adj. Stns. (4)	Double Area (2 x 4)
A	207	−331	−875	−181125
B	1309	− 98	898	+1175482
C	1508	567	875	+1319500
D	505	777	−898	−452592
			Algebraic Sum =	1861265 Sq.m.

Area of ABCD = $\frac{1}{2}$ x 1861265 = 930632 Sq. m.

Area of triangle AXD

= 930632 - 716572

= 214060 Sq. m.

Bearing of DA

$= \tan^{-1} \frac{\text{Dep. of A - Dep of D}}{\text{Lat of A - Lat of D}}$

$= \tan^{-1} \frac{(-331 - 777)}{(207 - 504)}$

$= \tan^{-1} \frac{-1108}{-297}$

or S 75°00'W

Whole circle bearing of DA = 255°00'

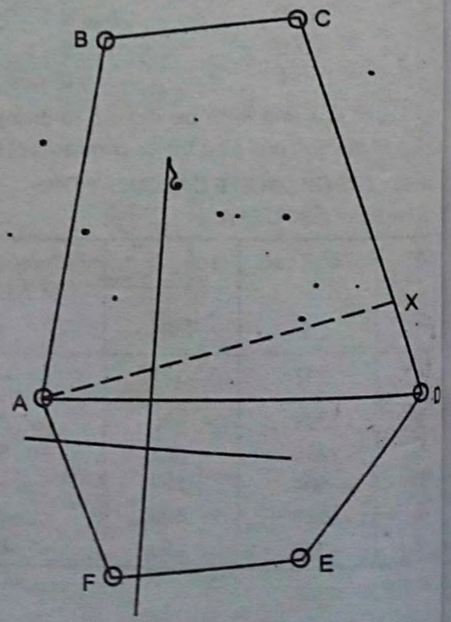
Length of DA

= 1108 Cosec 75° = 1147.1 m

Bearing of DC = $\tan^{-1} \frac{567 - 777}{1508 - 504}$

$= \tan^{-1} \frac{-210}{+1004}$

= N 11°49'W



Whole circle bearing of DC = 348°11'

Angle ADX = Angle ADC = Bearing of DC − Bearing of DA

= 348°11' - 255° = 93°11'

Area of triangle AXD = $\frac{1}{2}$DA x DX x Sin ADX

= 214060 = $\frac{1}{2}$ x 1147.1 x DX x Sin 93°11'

$\therefore$ DX = $\frac{2 \times 214060}{1147.1 \times \text{Sin } 93°11'}$ = 373.80 m

Co-ordinates of DX

Latitude = 373.80 x Cos 11°49' = 365.88 m.

Departure = −373.80 x Sin 11°49' = −76.55 m.

$\therefore$ **Co-ordinates of X : Latitude** = Lat of D + Lat of DX

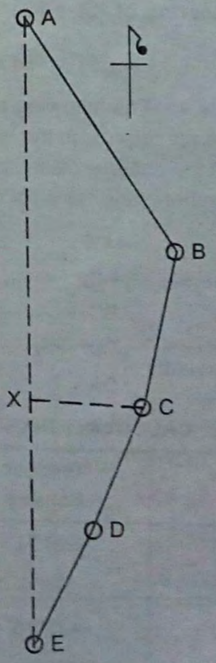
= 504.00 + 365.88 = **869.88 m.**

Departure = Dep. of D + Dep. of DX

= 777.00 + (−76.55) = 700.45 m.

Bearing of AX = $\tan^{-1} \frac{700.5 - (-331)}{869.9 - 207}$

$= \tan^{-1} \frac{1031.5}{662.9}$ = **N 57°16'E**



Example 48. The following are thenotes of an underground traverse :

Line	Azimuth	Hor. Distance
AB	150°07'30"	481.70 m
BC	187°58'00"	180.35 m
CD	192°31'20"	276.37 m
DE	188°34'00"	318.05 m

If X is the mid-point of the straight road joining A and E, calculate the bearing and horizontal length of a road to be driven from C to X.

Answer : TRAVERSE CALCULATION

Figure shows the relative positions of the stations, from which it will be seen that the length and bearing of AE must be calculated and the co-ordinates of X.

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Co-ordinates of Ea from A are calculated and tabulated below :

Line	Bearing	Distance m	Partial Co-ordinates Latitude	Partial Co-ordinates Departure	Total Co-ordinates Latitude	Total Co-ordinates Departure	Station
					0.00	0.00	A
AB	S 29°52'30"E	481.70	–417.68	+239.93	–417.68	+239.93	B
BC	S 7°58'00"W	180.35	–178.61	–25.00	–596.29	+214.93	C
CD	S 12°31'20"W	276.37	–269.79	–59.92	–866.08	+155.01	D
DE	S 8°34'00"W	318.05	–314.50	–47.38	–1180.58	+107.63	E
Total Co-ordinates of E =			–1180.58	+107.63			

Total co-ordinates of X :

Departure = E 107.63 ÷ 2 = E 53.815

Latitude = S 1180.58 ÷ 2 = S 590.29

Length and bearing of CX :

Bearing of CX $= \text{Tan}^{-1} \dfrac{214.93 - 53.815}{596.29 - 590.29} =$ N 87°52'02"W

Length = 161.115 cosec 87°52'20" = 161.54 m.

Example 49. The following are the lengths and bearings of the sides of a traverse ABCD, the bearings are referred to the magnetic meridian. The value of magnetic declination being 50°30'W. Convert the observed bearings to the true bearings. Calculate the co-ordinates with respect to true meridian. Find the error of closure.

Line	Length	Bearing
AB	470 M	343°52'
BC	635 M	87°50'
CD	430 M	172°40'
DA	563 M	265°12'

Answer : CALCULATION OF CO-ORDINATES

Line	Magnetic Bearing	True Bearing	Quadrant Bearing
AB	343°52'	343°52' – 50°30' = 293°22'	N 66°38'W
BC	87°50'	87°50' – 50°30' = 37°20'	N 37°20'E
CD	172°40'	172°40' – 50°30' = 122°10'	S 57°50'E
DA	265°12'	265°12' – 50°30' = 214°42'	S 34°42'W

Co-ordinates of stations A, B, C and D with respect to true meridian are calculated as follows :

Stn. Line	Hor. dist.	Q Bearing	Partial Co-ordinates Lat.	Partial Co-ordinates Dep.	Total Co-ordinates Lat.	Total Co-ordinates Dep.
A					0.00	0.00
AB	470	N 66°38'W				
B			+186.41	–431.45	+186.41	–431.45
BC	635	N 37°20'E				
C			+504.90	+385.09	+691.30	–46.42
CD	430	S 57°50'E				
D			–228.92	+364.00	+462.38	+317.58
DA	563	S 34°42'W				
A			–462.87	–320.50	–0.49	–2.92

Error of closure $= \sqrt{(0.49)^2 + (2.92)^2} = 2.96$ m.

Example 50. The following survey was made with a theodolite, graduated clockwise.

Observed Angle	Angle of Inclination	Distance in M
	AB + 7°15'	AB 300.8
ABC 227°17'20"	BC + 3°35'	BC 381.7
BCD 292°53'35"	CD – 6°20'	CD 238.9
CDE 256°8'40"	DE – 18°10'	DE 531.2

Assume AB to be bearing due South, It is proposed to drive a roadway from points E to A. Find out the length of the roadway and its gradient and bearing.

Answer : LENGTH, GRADIENT & BEARING OF A ROADWAY

Horizontal lengths of the lines AB, BC, CD and DE are calculated, and the difference of level between A and E are calculated as follows :

Line	Distance L	Inclination α	Hor.length L x Cosθ	Vert. difference L x Sinθ
AB	300.8	+ 7°15'	298.40	+ 37.96
BC	381.7	+ 3°35'	380.95	+ 23.86
CD	238.9	– 6°20'	237.44	– 26.36
DE	538.2	– 18°10'	511.37	– 167.80

Difference of level between A and E = + 37.96 + 23.86 – 26.36 – 167.80 = –132.34.

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Quadrant Bearings

AB = Due South

BC = 227°17'20" – 180° = S 47°17'20" W

CD = (227°17'20" + 292°53'35") – 180° = 340°10'55" (W.C.B) = N 19°49'05"W

DE = (340°10'55" + 265°08'40") – 540° = 65°19'35" = N 65°19'35" E.

Calculation of co-ordinates :

Line Stn.	Horizontal Length	Quadrant Bearing	Partial Co-ordinatess		Total Co-ordinates	
			Latitude	Departure	Latitude	Departure
A					0.00	0.00
AB	298.40	Due South				
B			–298.40	0.00	–298.40	0.00
BC	380.95	S 47°17'20"W				
C			–258.40	–279.90	–556.80	–279.90
CD	237.44	N 19°49'05"W				
D			+223.38	–80.50	–333.42	–300.40
DE	511.37	N 65°19'35"E				
E			+213.47	+464.68	–119.95	+104.28

Horizontal length of the roadway AE = $\sqrt{(119.95)^2 + (104.28)^2}$

= **158.94 metres.**

Gradient of the roadway = 132 m in 158.94 m = **1 in 1.201**

Bearing of the roadway AE = $\tan^{-1} \frac{+104.28}{-119.95}$ = S 41°00'09"E

Hence **bearing of EA = N 41°00'09"W**

Example 51. The haulage roadways from two districts of a flat seam of a colliery are to be driven along DC and BC to join a main haulage roadway at C. The roadways DC and BC are to be 1250 m and 1060 m respectively. The co-ordinates of the traverse connecting B and D are as follows :

No.	Line	Length	Bearing	Latitude	Departure
1	AB	725.7 m	S 59°45'E	-365.26	+626.24
2	BC	1060 m	?		
3	CD	1250 m	?		
4	DE	945 m	S 55°18'W	–537.99	–776.92
5	EA	577.2 m	S 2°40'W	–576.63	–26.85

Calculate the bearings on which the roadways DC and BC should be driven.

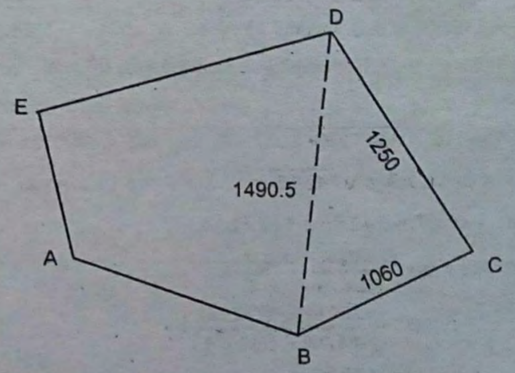
Answer : BEARING OF ROADWAYS

Total co-ordinates of B with respect to D are calculated as follows :

No.	Line	Length	Bearing	Partial Co-ordinates		Total Co-ordinates		Station
				Latitude	Departure	Latitude	Departure	
						0.00	0.00	D
4	DE	945	S 55°18'W					
				–537.99	–776.92	–537.99	–776.92	E
5	EA	577.2	S 2°40'W					
				–576.63	–26.85	–1114.62	–803.77	A
1	AB	725.7	S 59°45'E					
				–365.26	+626.24	–1479.88	–177.53	B

Bearing of line DB = $\tan^{-1} \frac{-177}{-1479.88}$ = S 6°50'30"W

Length of line DB = $\sqrt{1479.88^2 + 177.53^2}$ = 1490.5 m



In triangle DBC, all three sides are known and the value of angles CDB and DBC are calculated as follows :

$$\text{Cos CDB} = \frac{1250^2 + 1490.5^2 - 1060^2}{2 \times 1250 \times 1490.5} = 0.7139859$$

$\angle CDB = \text{Cos}^{-1} 0.7139859 = 44°26'23"$

Bearing of DC = Bearing of DB - 44°26'23"

= 186°50'30" – 44°26'23" = 142°24'07"

= S 37°35'53"E.

$$\text{Similarly, Cos DBC} = \frac{BC^2 + CB^2 - DC^2}{2 \times BD \times CB} = \frac{1490.5^2 + 1060^2 - 1250^2}{2 \times 1490.5 \times 1060}$$

DBC = $\text{Cos}^{-1} 0.5641674 = 55°39'20"$

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Bearing of BC = Bearing of BD + 55°39'20" = 6°50'30" + 55°39'20"

= N 62°29'50"E.

The roadways BC and DE are to be driven on bearings of N 62°29'50"E and S 37°35'53"E respectively.

Example 52. The co-ordinates in metres of centres of two shafts A and B and their respective depths and surface levels are as given below :

Shaft	*Lat*	*Dep*	*Depth*	*Surface level*
A	N 669.03	W 237.44	182.88	249.94 A.D.
B	N 109.73	E 250.24	137.18	163.37 A.D.

Determine the length, bearing and gradient of a cross-measure dfirt to connect the bottom of shafts A and B.

Answer : CROSS-MEASURE DRIFT PROBLEM

The difference in level between bottom of two shafts is calculated as follows -

Shaft	*Surface Level*	*Depth*	*R.L.at bottom of shaft*
A	(249.94m A.D.) – (182.88m) =		67.06m
B	(163.37m A.D.) – (137.18m) =		26.19m

Difference in level between bottom of shafts A and B = 67.06 - 26.19 = 40.87 metres.

Horizontal length,

$$AB = \sqrt{(669.03 - 109.73)^2 + (-237.44 - 250.24)^2} = 742.05 \text{ metres.}$$

Inclined length of drift, $AB = \sqrt{742.05^2 + 40.87^2}$ **= 743.17 metres.**

Gradient of drift = 40.87 in 742.05 = **1 in 18.16.**

Bearing of drift, $AB = \tan^{-1} \dfrac{\text{Dep.A} - \text{Dep.B}}{\text{Lat. A} - \text{Lat. B}}$

$$= \tan^{-1} \frac{(-237.44) - (250.24)}{(669.03) - (109.73)}$$

$$= \tan^{-1} \frac{-487.68}{559.30} = \textbf{S 41°05'12"E.}$$

Example 53 : Following are the survey notes of closed traverse ABCDE together with latitudes and departures of the known sides :

Line	Length (m)	Bearing	Latitude	Departure
AB	217.50	S 59°45' E	109.578	187.872
BC	x	N 62°32' E	x	x
CD	x	N 37°36' W	x	x
DE	283.50	S 55°18' W	161.397	233.076
EA	173.15	S 2°40' W	172.989	9.055

The lengths of BC and CD could not be measured. Calculate their lengths.

Answer : LENGTHS OF ROADWAYS

Total Co-ordinates of B with respect to D are calculated as follows :

Line Stn.	Length (m)	Bearing	Partial Co-ordinate Lat	Partial Co-ordinate Dep	Total Co-ordinate Lat	Total Co-ordinate Dep
D					0.00	0.00
DE	283.50	S 55°18' W				
E			– 161.391	– 233.078	– 161.391	– 233.078
EA	173.15	S 2°40' W				
A			– 172.962	– 8.055	– 334.353	– 241.133
AB	217.50	S 59°45' E				
B			– 109.570	+ 187.884	– 43.932	+ 53.249

Bearing of line DB = $\tan^{-1} \dfrac{-53.249}{-443.923}$ = S 6°50'24"W

Length of line DB = $\sqrt{(443.923)^2 + (53.249)^2}$

= 447.105 m.

In triangle BDC, bearing of DB = S 6°50'24" W and length of BD = 447.105 m.

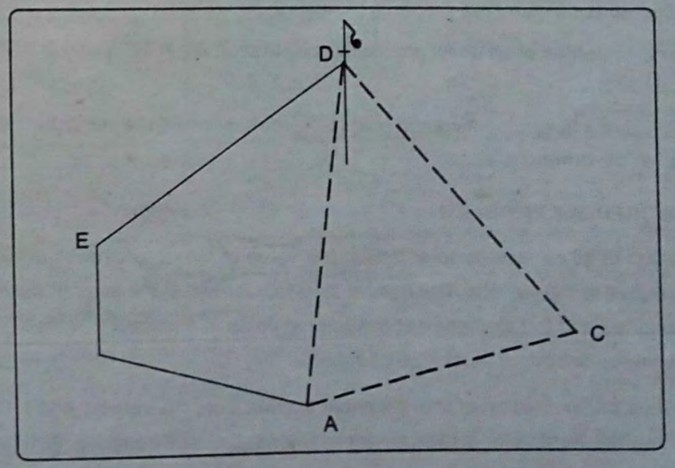
∠DBC = Bearing of BC – Bearing of BD

= 62°32' – 6°50'24" = 55°41'36"

∠BDC = Bearing of DC – Bearing of DB

= 37°36' + 6°50'24" = 44°26'24"

∠DCB = 180° – (55°41'36" + 44°26'24") = 79°52'



Applying sine rule, we can find out the lengths of DC and BC.

Length of DC :

$$\frac{DC}{\text{Sin } B} = \frac{BD}{\text{Sin } C} \qquad \therefore DC = \frac{447.105 \times \text{Sin } 55°41'36"}{\text{Sin } 79°52'}$$

$$= 375.175 \text{ m.}$$

Length of BC :

$$\frac{BC}{\text{Sin } D} = \frac{BD}{\text{Sin } C} \qquad \therefore BC = \frac{447.105 \times \text{Sin } 44026'24"}{\text{Sin } 79052'}$$

$$= 318.009 \text{ m.}$$

Example 54. Following are the records taken from a borehole survey :

Section (m)	Inclination (Degrees)	Bearing
0 to 45	89°30'	N 23° E
45 to 160	89°15'	N 27° E
160 to 280	88°00'	N 30° E
280 to 370	85°00'	N 34° E
370 to 475	87°15'	N 32° E
475 to 520	87°00'	N 33° E

The co-ordinates of the centre of borehole at the surface are N 1656 and E 3420 and its R.L. is 160 m above M.S.L.

The co-ordiantes of an underground Survey Station are N 1775 and E 3770 and its R.L. is 366 m below M.S.L.

Calculate the length, bearing and gradient of the line joining the underground Survey Station to the base of the borehole.

Answer : BOREHOLE PROBLEM

It is first of all necessary to establish the value of the co-ordinates at the bottom of the borehole relative to the surface. The horizontal displacement at the base of each survey section are first obtained and the displacements summed up as in a normal traverse. The true vertical distance from top to bottom must also be found.

Let θ be the inclination of the borehole section from the vertical and L the length of the section. Then the horizontal displacement of the section (D) equals L Sin θ and the vertical equivalent equals L Cos θ .

The vertical equivalent and the horizontal displacement of sections are calculated as follows :

Borehole Section		Length of Borehole	Inclination	Deviation from Vertical	Vertical Equivalent	Horizontal Displacement
From	To	L mts.		θ	L cos θ	L sin θ
0	45	45	89°30'	0°30'	44.998	0.397
45	160	115	89°15'	0°45'	114.990	1.505
160	280	120	88°00'	2°00'	119.927	4.188
280	370	90	85°00'	5°00'	89.657	7.844
370	475	105	87°15'	2°45'	104.879	5.038
475	520	45	87°00'	3°00'	44.938	2.355

Total vertical equivalent of boreholes = 519.389 metres

The R.L. of the borehole bottom = 519.389 – 160

= 359.389m B.O.D.

The co-ordinates of the borehole are calculated as follows :

Line	Horizontal displacement mts.	Bearing	Latitude mts.	Departure mts.
1	0.397	N 23°E	0.365	0.155
2	1.505	N 27°E	1.340	0.683
3	4.188	N 30°E	3.627	2.094
4	7.844	N 34°E	6.503	4.386
5	5.038	N 32°E	4.272	2.670
6	2.355	N 33°E	1.975	1.283
			N 18.082	E 11.271

Co-ordinates of the top of the borehole = N 1656.00 E 3420.000

Co-ordinates of the base of the borehole relative to the top = **N 18.082 E 11.271**

Therefore co-ordinates of the bottom of the borehole = N 1674.082 E 3431.271

Co-ordinates of underground station N 1775 E 3770

Co-ordinates of base of borehole N 1674.082 E 3431.271

Differences 100.918 338.729

The **bearing of the line** from the underground station to the base of the borehole = $\tan^{-1}\frac{338.729}{100.919}$ SW = **S73°27'34"W**

Horizontal length of the line $= \sqrt{338.729^2 + 100.919^2}$

$= 353.443$ metres.

Difference of level $= 366 - 359.389 = 6.611$ metres

Gradient $= 6.611$ m in 353.443 m $= 1$ in 53.463.

Inclined length $= \sqrt{353.443^2 + 6.611^2} = 353.505$ metres.

Example 55. The co-ordinates of three trigonometric stations A, B and C are –

Station	Latitude (m)	Departure (m)
A	0	0
B	60N	387E
C	178S	598E

The angles measured at a colliery station P are

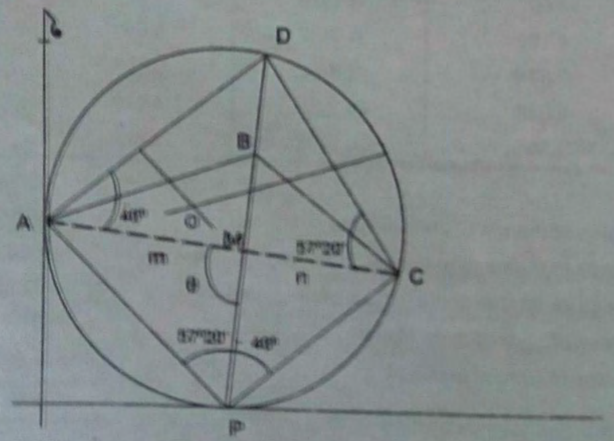
APB = 57°20′ BPC = 46°00′

Calculate the Co-ordinates of station P.

Answer : THREE POINT PROBLEM

The co-ordinate positions of stations A, B and C are roughly plotted. The angles ACD = 57°20′ and CAD = 46° are set out. The perpendicular bisectors of AD and CD are intersected at a point O. A circle is drawn through the points A, C and D by taking O as centre of the circle.

DB is joined and it is produced to meet the circumference of the circle at P. P is the position of the observing station. Angles APD and ACD are on the same arc AD subtending angles at the circumference of the circles. Therefore, angles APD = ACD, and similarly angles DPC = DAC.



The two lines AC and BP meet at point M. Let AM = m and CM = n. Angles AMP = BMC = θ.

From the given co-ordinates :

Bearing of AB $= \tan^{-1}\frac{+387}{+60} =$ N 81°11′13″E

Bearing of BC $= \tan^{-1}\frac{+(598-387)}{-(187+60)} =$ S 41°33′31″E

Bearing of AC $= \tan^{-1}\frac{+598}{-178} =$ S 73°25′26″E

∴ Distance AC = 178 Sec S 73°25′26″ = 623.93 metres.

From the above bearings :

Angle BAC = (90° – bearing of AB) + (90° – bearing of AC)

= (90° – 81°11′13″) + (90° – 73°25′26″)

= 25°23′21″

Angle ABC = 81°11′13″ + 41°33′31″ = 122°44′44″

Angle BCA = 180° – (25°23′21″ + 122°44′44″) = 31°41′55″

We know the trigonometrical relations :

$(m + n)\cot\theta = m\cot\alpha - n\cot\beta$, and

$(m + n)\cot\theta = n\cot\beta_1 - m\cot C_1$

From tringle ABC :

$(m + n)\cot\theta = n\cot BAC - m\cot BCA$

$= n\cot 25°23'21'' - m\cot 31°41'55''$.

From tringle APC :

$(m + n)\cot\theta = n\cot CPB - m\cot APB$

$= n\cot 46° - m\cot 57°20'$.

From the above relations :

$n\cot 25°23'21'' - m\cot 31°41'55'' = n\cot 46° - m\cot 57°20'$.

$$\text{or, } \frac{m}{n} = \frac{\cot 25°23'21'' - \cot 46°}{\cot 31°41'55'' - \cot 57°20'} = 1.1669389$$

or $m = 1.1669389n$; But $m + n = AC = 623.93$ metres.

$\therefore 1.1669389n + n = 2.1669389n = 623.93$ metres.

$$n = \frac{623.93}{2.1669389} = 287.93151 \text{ metres.}$$

$m = 623.93 - 287.93151 = 335.99849$ metres.

Putting the values of m and n, in $(m + n)\cot\theta$

$= n\cot 46° - m\cot 57°20'$, we get

$$\text{Cot}\,\theta = \frac{287.9351\ \text{Cot}\ 46^\circ - 335.99849\ \text{Cot}\ 57^\circ 20'}{623.93}$$

$$= 0.1003653$$

$$\theta = 84^\circ 16'07''$$

Angle PAM = $180^\circ - (84^\circ 16'07'' + 57^\circ 20') = 38^\circ 23'53''$.

$$AP = \frac{335.99849 \times \text{Sin}\ 84^\circ 16'07''}{\text{Sin}\ 57^\circ 20'} = 397.13582\ \text{m}.$$

Bearing of AP = $73^\circ 25'26'' - 38^\circ 23'53'' = S\ 35^\circ 01'33''E$.

Now Co-ordinates of P in relation to local origin A –

Latitude = $397.13582 \times \text{Cos}\ 35^\circ 01'33''$

= S 325.212 m

Departure = $397.13582 \times \text{Sin}\ 35^\circ 01'33''$

= E 227.9344 m

Example 56. Discuss the advantages of co-ordiante plotting over protractor plotting. The co-ordinates of three points A, B and C are–

Stn.	Latitude	Departure
A	110 N	480 W
B	171 N	192 E
C	40 S	62 W

The angle observed at a colliery station P are–

∠APC = 34°12' ; ∠CPB = 41°18'. Calculate the co-ordiantes of station P.

Answer : ADVANTAGES OF CO-ORDINATE PLOTTING

Advantages of Co-ordinate plotting over protractor plotting are–

1. It enables a survey to be plotted with great precision.
2. Each point may be plotted independently of any other point in the survey.
3. The bearing and length of the closing line may be calculated to any required degree of accuracy consistent with the accuracy of the instrumental work.
4. Accuracy of work can be checked before plotting a closed traverse.
5. The area of any closed traverse may be calculated directly from the known co-ordinates of the angular points.
6. Shrinkage of plan at any part can be ascertained readily from the co-ordinate graticules or squares.
7. Plotting of large survey can proceed simultaneously on different sheets or on different parts of sheet.
8. The computation form is available for subsequent checking.

On the otherhand, plotting by protractor is a simple and convenient process, but only

limited degree of accuracy is obtainable, depending on the scale of the plan, the smallest division on the protractor and the skill of the surveyor. Moreover, unless the survey is a closed one, no check on the plotting is possible, and if the bearing of the closing line is desired this can only be obtained approximately.

Three point Problem :

Bearing of AC $\tan\theta = \frac{418}{150}$ $\theta = 70^\circ 16'$

Bearing of CB $\tan\alpha = \frac{254}{211}$ $\alpha = 51^\circ 37'$

$$\text{Length of AC} = \frac{418}{\text{Sin}\ 70^\circ 16'} = 444.1\ \text{metres}.$$

$$\text{Length of BC} = \frac{354}{\text{Sin}\ 57^\circ 37'} = 324.1\ \text{metres}.$$

$$PC = \frac{\text{Sin A} \times AC}{\text{Sin}\ 34^\circ 12'} = \frac{\text{Sin B} \times BC}{\text{Sin}\ 41^\circ 18'}$$

$$\frac{\text{Sin A}}{\text{Sin B}} = \frac{324.1}{444.1} \times \frac{\text{Sin}\ 34^\circ 12'}{\text{Sin}\ 41^\circ 18'} = 0.6215$$

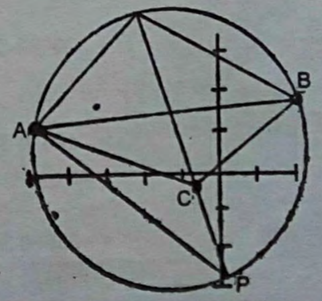
Angle BCA = $360^\circ - (70^\circ 16' + 51^\circ 37') = 238^\circ 07'$

A + B = $360^\circ - (238^\circ 07' + 34^\circ 12' + 41^\circ 18') = 46^\circ 23'$

$A = 46^\circ 23' - B$.

$$\frac{\text{Sin}\ (45^\circ 23' - B)}{\text{Sin B}} = 0.6215 \text{ or, } \text{Cot B} = \frac{0.6215 + \text{Cos}\ 46^\circ 23'}{\text{Sin}\ 46^\circ 23'}$$

therefore $B = 28^\circ 54'$ and $A = 17^\circ 29'$.



$$\text{Length AP} = \frac{AC\ \text{Sin}\ 51^\circ 41'}{\text{Sin}\ 34^\circ 12'} = \frac{44.11 \times \text{Sin}\ 51^\circ 41'}{\text{Sin}\ 34^\circ 12'} = 619.8\text{m}$$

Bearing of AP = $S(\theta - A)\ E = S\ (73^\circ 16' - 17^\circ 29')\ E = S\ 52^\circ 47'E$.

Partial co-ord. of P :

Latitude = 619.8 Cos 52°47'S = 375 S

Departure = 619.8 Sin 52°47' E = 493.7 E

Total co-ordinates of P are 265 South : 13.7 East.

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Example 57. ABCDEF represents the boundary between two properties. The co-ordinates of points A to F are given below. Calculate the length and bearing of the line which will straighten the boundary between the two properties. (Assume the equalizing line passes through A).

Station	Latitude	Departure
A	302	358
B	506	390
C	158	496
D	368	560
E	279	620
F	393	700

Answer : LENGTH & BEARING OF EQUALIZING LINE

The boundary ABCDEF is roughly plotted. Let AA' and FF' represent the closing latitudes and A'F' the east-west axis with zero latitude. Now the area of the figure A'ABCDEFF' is calculated as follows :

Area of A'ABB'

$$= \frac{(AA' + BB')}{2} \times A'B' = \frac{302 + 506}{2} \times (390 - 358) = 12928 \text{ Sq.m.}$$

Area of B'BCC'

$$= \frac{506 + 258}{2} \times (496 - 390) = \frac{764}{2} \times 106 = 40492 \text{ Sq.m.}$$

Area of C'CDD'

$$= \frac{258 + 368}{2} \times (560 - 496) = \frac{626}{2} \times 64 = 20032 \text{ Sq.m.}$$

Area of D'DEE'

$$= \frac{368 + 279}{2} \times (620 - 560) = \frac{647}{2} \times 60 = 19410 \text{ Sq.m.}$$

Area of E'EFF'

$$= \frac{279 + 393}{2} \times (700 - 620) = \frac{672}{2} \times 80 = 26880 \text{ Sq.m.}$$

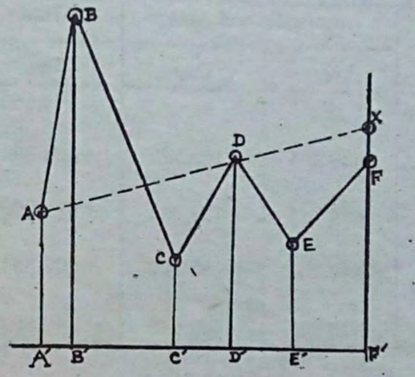
Total Area = 119742 Sq.m.

Let the equalizing line AX intersects the line running North and South through F at the point X. The condition for AX to straighten the boundary is that the area of the trapezium AXF'A' must be equal in area to the figure A'ABCDEFF'.

A'F' = 700 – 358 = 342 metres. Let F'X is 'X' metres. Then the area, $AXF'A' = \frac{302 + x}{2} \times 342$ Sq. metres.

But the equalised area, $AXF'A' = \frac{302 + x}{2} \times 342 = 119742$ or x = 398.25 metres.

Bearing of AX $= \tan^{-1} \frac{700 - 358}{398.25 - 302} = \tan^{-1} \frac{342}{96.25} =$ **N 74°17'E**



Length of AX $= \sqrt{(700 - 358)^2 + (398.25 - 302)^2} =$ **355.286m.**

The point X is situated (398.25 – 393.00) = 5.25 metres North of **F**.

Example 58. The following are the lengths and bearings of the sides of a traverse ABCD. The bearings are referred to the magnetic meridian, the value of declination being 5°30'W. Convert the observed bearings to true bearings and find the error of closure.

Line	Length (m)	Bearing (Magnetic)
AB	470.00	343°52'
BC	635.00	87°50'
CD	430.00	172°40'
DA	563.00	263°12'

Answer : ERROR OF CLOSURE

Line	Magnetic Bearing	Declination	True Beaing
AB	343°52'	5°30'W	343°52' – 5°30' = 338°22'
BC	87°50'	5°30'W	87°50' – 5°30' = 82°20'
CD	172°40'	5°30'W	172°40' – 5°30' = 167°10'
DA	263°12'	5°30'W	263°12' – 5°30' = 257°42'

Co-ordinates of stations A, B, C and D with respect to true meridian are calculated as follows :

Stn. Line	Hor. dist (m)	Bearing	Partial Co-ordinates		Total Co-ordinates	
			Lat.	Dep.	Lat.	Dep.
A					0.000	0.000
AB	470	338°12'				
B			+ 436.894	– 173.272	+ 436.894	– 173.272
BC	635	82°20'				
C			+ 84.715	+ 629.323	+ 521.609	+ 456.051
CD	430	167°10'				
D			– 419.258	+ 95.509	+ 102.351	+ 551.560
DA	563	257°42'				
A			– 119.936	– 550.076	– 17.585	+ 1.484

Error of closure = $\sqrt{(-17.585)^2 + (+1.484)^2}$ = **17.647 m.**

Example 59. Following are the co-ordiantes of a five sided polygon. Calculate the area in hectares enclosed by them.

Stations	Total Latitude (m)	Total Departure (m)	Remarks
A	0.00	0.00	
B	– 560.25	+ 315.73	
C	– 211.93	+ 1000.00	
D	+ 200.52	+ 980.35	
E	+ 640.90	0.00	

Answer : AREA OF A CLOSED TRAVERSE

Stn.	Total Latitude (m)	Total Departure (m)	Sum of Latitudes (m)	Diff of Departures (m)	Double Area (Sq. m)	
					+ve	–ve
A	0.00	0.00	– 560.25	– 315.73	176887.73	
B	– 560.25	+ 315.73	– 772.18	– 684.27	528379.61	
C	– 211.93	+ 1000.00	– 11.41	+ 109.65		1251.11
D	+ 200.52	+ 890.35	+ 841.42	+ 890.35	749158.30	
E	+ 640.00	0.00	+ 640.90	0.00	0.00	
					1454425.64	1251.11
					1251.11	
				Double Area	1453174.53	Sq. m.

Area of polygon = 1453174.53 ÷ 2 = 726587.27 Sq. m.

Area in hectares = 726587.27 ÷ 10000 = **726.58 Hectares.**

PROBLEMS

Problem 1. Plot the following survey by co-ordinates and from the latitudes and departures, calculate the length and bearing of the closing line :

Line	*Meridian Bearing*	*Distance*
AB	25°0'	130m
BC	70°30'	320m
CD	120°0'	340m
DE	210°30'	410m

[Ans. 534.21m, S 56°00'46"E]

Problem 2. The following are the notes of a closed traverse :

Line	*Azimuth*	*Distance*
AB	43°30'	43m
BC	Closing line	
CD	250°00'	47m
DE	290°00'	78m
EA	65°00'	75m

Plot the traverse on a suitable scale and ascertain by measurement the azimuth and length of BC.

[Ans. 165°, 76m]

Problem 3. The galleries BA and EF are being driven to meet each other. The notes of traverse run from A to F are given below :

Line	*Distance*	*Bearing*
AB	450m	S 60°E
BC	350m	N 25°E
CD	300m	N 23°W
DE	320m	N 85°W
EF	300m	S 34°W

If G be the point of intersection of BA and EF, calculate the length AG and FG by which the galleries are to be extended to meet each other. Check the calculation by plotting.

[Ans. AG = 137.66m, FG = 94.94m]

Problem 4. The following traverse is run from A to F in a hilly country. It is proposed to drive a tunnel from C to F. Calculate the length and bearing of same :

Line	*Distance*	*Bearing*
AB	185m	5°45'
BC	142.5m	355°00'
CD	150m	15°15'
DE	165m	342°00'
EF	375m	225°00'

[Ans. 279.103m. N 82°29'30"W]

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Problem 5. The following are the details of a traverse survey between faces of two galler which are to be driven forward to meet. Calculate the distance still to be driven in each roadw

Line	*Azimuth*	*Distance*
AB	340°	250m
BC	80°	520m
CD	140°	340m
DE	230°	300m

[Ans. BA to F = 388.52m. DE to F = 368.71

Problem 6. The following traverse is run to set out a line AE 567 metres long at right angles given line AB.

Line	*Length (metres)*	*Bearing*	*Latitude (metres)*	*Departure (metres)*
AB		360°	-	-
AC	252	119°30'	–124.1	+219.3
CD	306	85°24'	+24.5	+304.8
DE	?	?	-	-

Calculate the required length and bearing of DE. **[Ans. 108.47m. N 23°18'10"E**

Problem 7. The following table gives notes of an underground traverse. Calculate the azimuth length and gradient of a drift to connect A and F.

Line	*Azimuth*	*Distance*	*Inclination*
AB	89°54'	107m	–
BC	150°12'	58m	–
CD	180°00'	183m	–
DE	140°18'	47m	+28°
EF	228°36'	244m	–12°

[Ans. S 2°15'41"W, 423.97m, 1 in 14.7

Problem 8. The following are the latitudes and departures in metres of a closed traverse ABCD

Line	*Latitude*	*Departure*
AB	–232.2	–88.8
BC	+13.6	+116.4
CD	+161.0	+34.4
DA	+57.6	+62.0

Plot the traverse on a scale of 50 metres to one inch and also calculate area of traverse. **[Ans. 18891.92 Sq. m.**

Problem 9. Given the following total co-ordinates in metres of certain points. Calculate the area of enclosed figure. Also plot the traverse.

A	S 450	W 132
B	S 118	E 138
C	N 726	W 217
D	N 360	E 683
E	S 768	E 523

[Ans. 7,35,635 Sq.m.]

Problem 10. Following are the co-ordinates in metres of five survey stations. Calculate the area enclosed by them.

Stn.	Line	Latitude	Departure
A	AB	–298	+169
B	BC	–151	+362
C	CD	+630	+383
D	DE	+301	–560
E	EA	–482	–354

[Ans. 6,99,879 Sq.m.]

Problem 11. The haulage roadways from two districts of a flat seam of a colliery are to be driven along DC and BC to join a main locomotive haulage roadway at C. The roadway DC and BC are to be 375 m and 318 m large respectively. The co-ordinates of the traverse connecting B and D are as follows :

No.	*Line*	*Length*	*Bearing*	*Latitude*	*Departure*
1	AB	217.71 m	S 39°45'E	–109.578	+187.872
2	BC	318.00 m	?	?	?
3.	CD	375.00 m	?	?	?
4.	DE	283.50 m	S 55'18"W	–161.397	–232.076
5.	EA	173.16 m	S 2°40'W	–172.989	–8.055

Calculate the bearings on which the roadways DC and BC should be driven.
[Ans. BC towards N 62°29'30"E and DC on S 37°35'30"E]

Problem 12. From a point E a line EF is to be set out parallel to a given line AB, and such that ABF is a right angle, the points E and F being invisible from A and B. This is done by running the following traverse :

Line	*Length in m*	*Bearing*	*Latitude*	*Departure*
BA	–	360°	–	–
BC	96.0	35°15'	+78.39	+55.38
BD	144.0	115°40'	-62.37	+129.78
DE	87.0	42°30'	+64.14	+58.77

Calculate the necessary lengths and bearings of EF and FB.
[Ans. EF = 80.16, 180°, FB = 243.93 m, 270°]

Problem 13. It was impossible to observe the length and bearing of the line PQ directly. The following observations were, therefore, taken from two stations A and B.

Line	*Length in m*	*Bearing*
AP	126.00	S 65°36'W
AB	314.40	N 24°12'E
BQ	115.50	N 76°48'W

Compute the length and bearings of PQ and angles APQ and BQP.
[Ans. N 19°45'E, 388.04, 45°51', 96°33']

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Problem 14. The following notes refer to part of a traverse survey -

Line	*Length in m*	*Bearing*
PQ	186.0	30°15'
QR	649.6	134°32'
RS	424.8	222°18'

Compute the distance between A point A on PQ, 112.0 m from P and a point B on PQ 288.0 m from R. **[Ans. AB = 677.2m**

Problem 15. The following are the latitudes and departures of the lines of a closed traverse ABCD

Line	*Length in m*	*Departure in m*
AB	-116.1	–44.4
BC	+ 6.8	+58.2
CD	+80.5	+17.2
DA	+28.8	–31.0

Compute the area of the traverse. **[Ans. 4723 Sq. m**

Problem 16. The notes of a traverse between two points A and E in a certain seam are as follows

Line	*Azimuth*	*Distance*	*Inclination*
AB	89°	180 m	+ 6°
BC	170°	135 m	- 30°
CD	181°	165 m	Level
DE	280°	106.5m	Level

It is proposed to drive a cross-measure drift from the point E to another point F exactly midway between A and B. Calculate the azimuth and length of EF.
[Ans. 359°33', inclined length = 266.52 m

Problem 17. Given the following co-ordinates in metres of certain points, calculate the area of enclosed figure :

B	N 100	E173.2
C	N 29.3	E 243.9
D	S 100.6	E 168.9
E	S 100.6	E 100.6
A	0.0	0.0

[Ans. 27835.5 Sq. metres

Problem 18. Calculate by co-ordinates the length and bearing of closing line AF from the following booking of traverse :

Line	*Magnetic Bearing*	*Length*
AB	105°30'	350 m
BC	55°45'	280 m
CD	305°00'	310 m
DE	280°30'	250 m
EF	210°00'	400 m

[Ans. 143.72 m; S 65°45'W

Problem 19. The following are the lengths and bearings of the sides of a traverse ABCD. The bearings are referred to the magnetic meridian. The value of magnetic declination being 50°30'W. Convert the observed bearings to the true bearings.Calculate the co-ordinates with respect to true meridian. Find the error of closure.

Line	*Length*	*Bearing*
AB	470 m	343°52'
BC	635 m	87°50'
CD	430 m	172°40'
DA	563 m	265°12'

[Ans. N 66°38'W, N 37°20'E, S 57°50'E, S 34°42'W, 2 987 m]

Problem 20. The following are the lengths and bearings of a part of a traverse ABCD :

Line	*Length*	*Bearing*
AB	650 m	340°18'
BC	1720 m	30°12'
CD	1150 m	140°30'

Calculate the length and bearing of line XY joining the points X and Y where X is on the line AB 580 m from A and Y is on the line CD 540 m from C.
[Ans. 1348.5 m, N 11°52'E]

Problem 21. An abstract from a traverse sheet for a closed traverse is given below :

Line	*Length*	*Latitude*	*Departure*
AB	200 m	–173.20	+100.00
BC	130 m	0.00	+130.00
CD	100 m	+86.60	+50.00
DE	250 m	+250.00	+0.00
EA	320 m	–154.90	+280.00

Balance the traverse by Bowditch's method. Calculate the enclosed area in hectares by any method. **[Ans. 6.0249 hectares]**

Problem 22. The following are the observations taken from two stations P and Q :

Line	*Length*	*Bearing*
PR	250.00 m	S 60°30'W
PQ	630.30 m	N 30°10'E
QS	190.70 m	N 84°48'W

Find the length and bearing of RS and the angles PRS and QRS.
[Ans. 767.00 m; 33°49', 111°29']

Problem 23. The following table gives lengths and bearing of the sides of a traverse ABCD. Compute the error of closure in magnitude and direction.

Line	*Azimuth*	*Distance*
AB	705.0 m	N 21°18'W
BC	952.5 m	N 82°20'E
CD	645.0 m	S 12°50'E

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DA	844.5 m	S 79°42'W

Find also the relative error of closure. **[Ans. 4.36 m, 54°32', 1/722]**

Problem 24. The following are the latitudes and departures of the lines of a closed traverse ABCD :

Line	*Lat (m)*	*Dep (m)*
AB	116.1S	44.4W
BC	6.8N	58.2E
CD	80.5N	17.2E
DA	28.8N	31.0W

Compute the area of the traverse. **[Ans. 4723 Sq. m]**

Problem 25. Two points A and D are connected by a traverse survey ABCD. The traverse is conducted in a counter-clockwise direction and the following measurements were recorded.

AB = 876m; BC = 682m; CD = 983m;

Angle ABC = 118°15'; BCD = 108°40'

Assuming that AB is in the meridian, find out the length AD and the angle BAD.

[Ans. 1420m; 68°12']

Problem 26. Two stations A and B are fixed on either side of a wood. In order to determine the length of AB, the following traverse is run :

Line	*Bearing*	*Length ih Metres*
AC	48°24'	438
CD	110°12'	664
DB	152°36'	582

Calculale the length of AB. An intermediate point E is to be fixed on AB running a line DE on a bearing of 168°06'. Detetmine the length of DE.

[Ans. 1301m; 462m]

Problem 27. A and B are two stations whose co-ordinates are as given below :

A	N 1056.9	E 585.1
B	N 1426.5	E 992.7

From A is run a line AC, 154.4m in length, on a bearing of 132°18' and from C is run a line CD, of length 544.0m, parallel to AB. Find the length and bearing of BD.

[Ans. 153.93 m, S 45°23'41"E]

Problem 28. The co-ordinates of the centres of two shafts A and B are respectively as follows :

Shaft A	N 658.5	W 233.7
Shaft B	N 108.0	E 246.3

The depths of the shafts and the level at the surface are respectively -

Shaft	Depths	Surface level
A	180m	246m A.O.D.
B	135m	160.8m A.O.D.

Determine the bearing, length and gradient of a cross measure drift to connect the bottom of shaft A to the bottom of shaft B. **[Ans. S 41°05'E, 731.48m; 1 in 18.17]**

Problem 29. It is required to find the distance between two inaccessible points C and D by observations from two stations A and B. The angular measurements are :

CAB 67°30', DAB 32°25', ABC 44°18' and ABD 72°41'.

Stations A and B are 584.5 metres apart.

[Ans. CD = 333.87 metres]

Problem 30. A roadway BA in a level seam is being driven to intersect a roadway EF at X. It is required to set off the line BA produced at point X in order to drive from X to meet BA. A Theodolite traverse has been carried out from A to F along an interconnecting roadway and details of this survey are :

Angle ABC = 230°15'	Length AB = 193.50 m
Angle BCD = 261°30'	Length BC = 135.00 m
Angle CDE = 235°15'	Length CD = 99.00 m
Angle DEF = 273°00'	Length DE = 315.00 m
	Length EF = 198.00 m

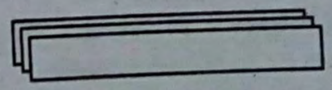
Calculate the distance along EF, from F, at which the line XAB will have to be set out and also the distance remaining to be driven from A to X.

[Ans. FX = 56.583m; AX = 74.178m]

Problem 31. A flat bauxite deposit of 15m average thickness has been located in a plateau region with a uniform laterite capping of 3m. Pillars ABCD and E closing at A define the mining lease and these pillars with reference to A are located as under :

Pillars	*Latitude*	*Departure*
B	– 984.81 m	+ 173.65 m
C	– 984.81 m	+ 1623.65 m
D	+ 609.10 m	+ 1484.20 m
E	+ 383.93 m	– 643.84 m

Compute the ore-reserves to the nearest million tonne and indicate the amount of overburden required to be stripped per tonne of ore. Assume specific gravity of bauxite = 2.5 and of laterite 2.8. **[Ans. ore = 76mt; 0.224 t per tonne]**



CHAPTER II

SETTING OUT CURVES

CURVES

Curve : Gradual and smooth change of direction from one straight to another is known as curve. The curves are generally circular arcs, but parabolic arcs are often used in some countries in lines of communication. The mine surveyor is frequently called upon to set out curves on surface or underground haulage roadway so as to overcome any obstacle intervening a straight path or in order to avoid derailment of train, side skidding, etc.

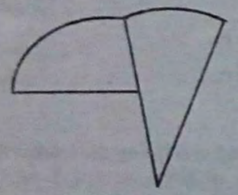
CLASSIFICATION OF CIRCULAR CURVES :

Circular curves are divided into three classes :

(a) Simple Curve
(b) Compound Curve
(c) Reverse Curve

Simple Curve : A simple curve maybe defined as a single circular curve uniting two tangents meeing at an angle. It consists of a single arc connecting two straights.

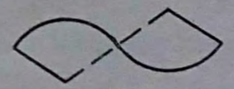
Simple Curve



Compound Curve

Compound Curve : A compound curve is one in which a curve of one radius runs into another of different radius bending in the same direction with a common tangent and with their centres on the same side.

Reverse Curve : It is also known as Ogee or Surpentine Curve. It is composed of two arcs of equal or different radius bending in opposite direction with a common tangent and with their centres on opposite sides of the curves.

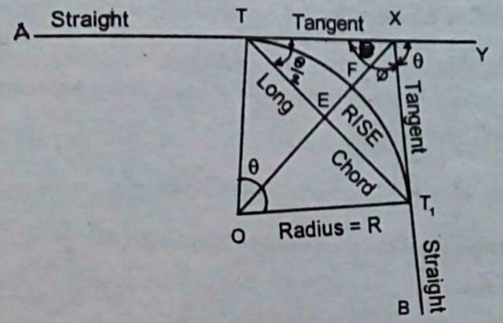


Reverse Curve

ELEMENTS OF SIMPLE CIRCULAR CURVES

The elements of a simple circular curve are shown in the figure, and the following definitions should be carefully noted :

(1) **Straights :** The two straight portions of railway line or roadway which are to be connected by a curve of radius R are the straights. AT and BT_1 in the figure are the two straights.



Element of Circular Curve

(2) **Tangent Points :** These are the ends of the curve where the alignment changes from a curve to a tangent. These are the beginning and end points of a curve. Points T and T_1 are tangent points.

(3) **Back Tangent :** The tangent AT previous to the curve is called the back tangent or first tangent.

(4) **Forward Tangent :** The tangent T_1B following the curve is called the forward tangent or second tangent.

(5) **Point of Intersection :** If the two tangents AT and BT_1 are produced, they will meet in a point, called the point of intersection or Vertex. The point X in the figure is the point of intersection.

(6) **Long Chord :** The chord joining the two tangent points is the long chord (TT_1).

(7) **Intersection Angle or Angle of Deflection :** The angle $YXT_1 = \theta$ between the tangents AX produced and XT_1 is called the intersection angle or the external angle of deflection between the two tangents. This angle (θ) is equal to the angle TOT_1 subtended at thecentre of curvature, O, by the arc of curvature TFT_1. The angle θ indicates the amount of deviation given by the curve.

(8) **Apex Angle :** It is the angle at the apex of the curve made by the two tangents. The apex angle $TXT_1 = \phi = 180^0 - \theta$.

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(9) **Tangent Distance :** It is the distance from the tangent point T or T_1 to the point of intersection.

(10) **Apex Distance :** The Distance FX from the mid-point of the curve to the point of intersection is the apex distance.

(11) **Rise :** The distance EF between the mid-point of the long chord to the mid-point of the curve is the rise of curve. It is also known as Middle ordinate or versed sine of the curve.

In practice, the radius of a curve is usually given and the angle of deflection $\theta = TOT_1$ can be calculated from the known direction of the two straights. It then frequently becomes necessary to calculate other elements of the curve, as follows -

To find the length of a Chord :

Let R be the radius of the curve and θ the angle subtended by the chord at the centre. Consider the long chord TT_1 in figure. Then -

$$\frac{ET}{OT} = \frac{\frac{1}{2}\text{ chord}}{R} = \text{Sin}\ \frac{\theta}{2} \qquad \therefore \text{Chord} = 2R\ \text{Sin}\ \frac{\theta}{2}$$

If the chord and the angle θ are known, Radius = Chord $\div$ 2R Sin $\frac{\theta}{2}$

To find the Tangent Distance :

Tangent distance = TX = XT_1. In the figure, we get,

$$\frac{TX}{OT} = \frac{\text{Tangent distance}}{\text{Radius}} = \tan\frac{\theta}{2}$$

$$\therefore \text{Tangent distance} = R \times \tan\frac{\theta}{2}$$

To find the Rise of the Curve = EF :

In figure, EF = FO – EO = Radius – EO, But $\frac{EO}{OT} = \text{Cos}\ \frac{\theta}{2}$

$$\therefore EO = R\ \text{Cos}\ \frac{\theta}{2}, \text{ and rise } EF = R - R\ \text{Cos}\ \frac{\theta}{2} = R\left(1 - \text{Cos}\ \frac{\theta}{2}\right)$$

To find the Apex Distance = XF :

$$XF = XO - \text{Radius. But } \frac{XO}{R} = \text{Sec}\ \frac{\theta}{2}, \text{ and } XO = R\ \text{Sec}\ \frac{\theta}{2}$$

$$\text{Apex distance} = XF = R\ \text{Sec}\ \frac{\theta}{2} - R = R\left(\text{Sec}\ \frac{\theta}{2} - 1\right)$$

To find the Length of Curve :

The length of the arc of a circle bears the same relationship to angle θ which it subtends at the centre of the circle as the whole circumference bears to 360°. Hence -

$$\frac{\text{Length of Curve}}{\theta} = \frac{2\pi R}{360} \qquad \therefore \text{Length of curve} = 2\pi R \times \frac{\theta}{360}$$

DESIGNATION OF CURVES

The sharpness of the curve is designated either by its *radius* or by its *degree of curvature*. The former system is adopted in Great Britain while the later system is used in America, Canada, India and some other countries.

The degree of curvature has slightly different definitions. According to the **arc definition** used in highway practice, the degree of the curve is defined as the central angle of the curve that is subtended by an arc of 30 m length. According to the **chord definition** generally used in railway practice, the degree of the curve is defined as the central angle of the curve that is subtended by its chord of 30 m length.

In metric system, two definitions for the degree of curve are in use :

1. Angle at the centre subtended by an arc (or chord) of 20 metres.
2. Angle at the centre by an arc (or chord) of 10 metres.

In 20 m arc (or chord) length is the basis for degree of the curve, we get -

$D^0 : 360^0 = 20 : 2\pi R$

From which $R = \frac{1145.92}{D} = \frac{1146}{D}$ metres (approx.)

If the definition is based on 10 m arc length, we have -

$D^2 : 360^0 = 10 : 2\pi R$

or, $R = \frac{573}{D}$ metres.

If 30 m chord length is used, $R = \frac{1719}{D}$ m [approx.]

SETTING OUT SIMPLE CURVES

Curves may be set out in various ways depending on the location of the curve, its length, the degree of accuracy required, the instruments available, the presence of obstacles, and so on.

Depending upon the instruments used, the methods of setting out curves may be divided into two classes-

1. **Linear Methods :** In the linear method, only a chain or tape is used. Linear methods are used when a high degree of accuracy is not required and the curve is short.

2. **Angular Method :** In angular method, an instrument such as a theodolite is used with or without a chain or tape.

Before a curve is set out, it is essential to locate the tangents, point of intersection, point of curve and point of tangency.

LOCATION OF TANGENT : Before setting out the curve, the surveyor is always supplied with a working plan upon which the general alignment of tangents is shown in relation to the traverse controlling the survey of that area. Knowing the offsets to certain points on both the tangents, the tangents can be staked on the ground by taping. The tangents may then be set out by theodolite by trial and error so that they pass through the marks as nearly as possible. The total deflection angle (θ) can then be measured by setting the theodolite on the point of intersection.

Location of Tangent points : After having located the point of intersection and measured θ, the tangent length can be calculated from the equation : $XT = R \tan\frac{\theta}{2}$

The point T can be located by measuring back a distance XT on the rear tangent. Similarly, the point T_1 can be located by measuring distance XT_1 on the forward tangent. The tangent points must be located with greater precision.

LINEAR METHODS OF SETTING OUT CURVES

Following are some of the linear methods of setting out simple circular curves -

1. By chords and offsets- outside the curve.
2. By tangents and offsets.
3. By Krohnke's tangential system.
4. By Jackson's six-point equidistant system.
5. By the versin method (successive bisection of arcs).
6. By offsets or ordinates from the long chord.

INSTRUMENTAL METHOD OF SETTING OUT CURVES : The following are the instrumental methods usually employed for setting out a circular curve -

(a) By Chords and angles.
(b) By Rankine's method of Tangential Angles.
(c) By two theodolites method.

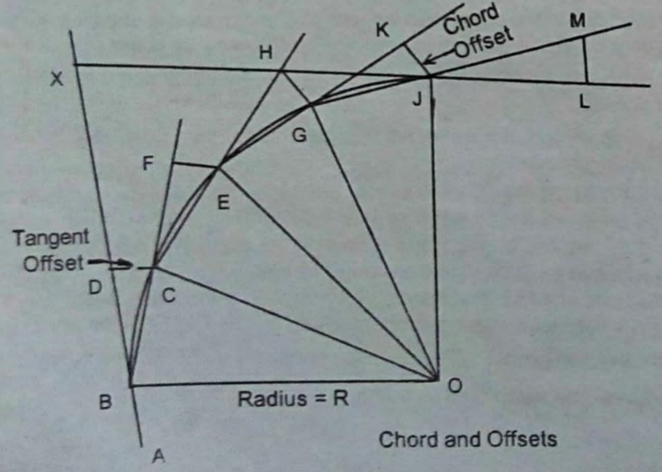
SETTING OUT BY CHORDS AND OFFSETS : In this method, the curve is set out by measuring offsets from the continuations of the straight AB and the successive chords BC, CE, EG etc. No angular instrument is required.

The Method consists of dividing the curve into a number of equal arcs, BC, CE_1 etc. each subtending an angle α at the centre, and to calculate the chord lengths from the usual formula,

$$\text{Chord} = 2R \sin \frac{\alpha}{2}$$

To find the first point C, pegs at A and B are placed and ranged them through D. Then DC is to be set out at right angles to BD, so that -

$$\text{Tangent Offset} = DC = \frac{\text{Chord}^2}{2R}$$



Chord and Offsets

If the radius of the curve is 300 metres and the chord is 30 metres :

$$\text{Tangent offset} = \frac{30^0}{2 \times 300} = \frac{900}{600} = 1.5 \text{ m}$$

$$\text{Chord offset} = FE \frac{\text{Chord}^2}{R} = \frac{30^2}{300} = 3 \text{ m}$$

Note that all the chord offsets, upto and including KJ, are double the tangent offset.

To set out the curve, the zero of a tape is held on the tangent peg at B and a survey pin (arrow) put in a long the tangent line at the point D so that in the triangle BCD, the length BC equal to 30 metres, the length of the offset DC, tangent offset, is 1.5 metres and DC is at right angles to BD.

Therefore the length BD will be $\sqrt{30^2 - 1.5^2} = 29.962$ m. The offset line DC is set off at right angles for a distance of 1.5 m which should fix the peg C exactly 30 m from the tangent point B. The tape is now moved forward and its zero end held on peg C whilst its front end is ranged in line with C and B. A survey pin (arrow) is put in at F and the tape moved inwards to point E making the distance FE equal to 3 m. The succeeding pegs are set out in like manner until the other end of the curve is reached.

If the closing point J (the second tangent point) and the straight JL are already fixed on the ground, the offset KJ will not be needed except as a check.

If the curve is not divided into exact number of arcs, owing to the use of fixed length with the result that chord GJ is a sub-chord (i, E, shorter than the others) the length of the chord offset KJ must be found independently from the formula, chord offset = $\frac{\text{Chord}^2}{R}$.

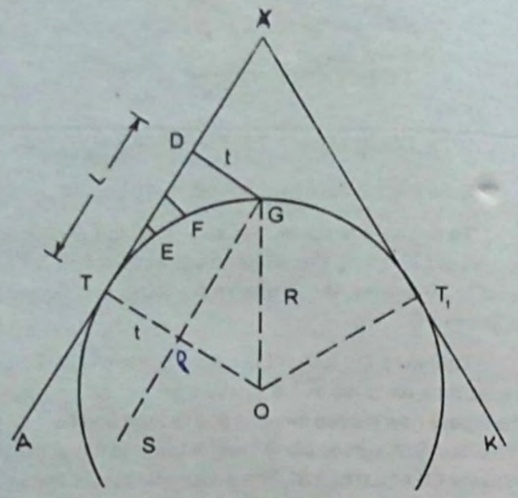
If the second straight JL is not ranged out independently, but has to be lined out as a continuation of the curve (as when driving a curve underground) it will be necessary to use the final tangent offset ML, equal in length to DC, and set out from the prolongation of the last complete chord GJ. If the chord GJ is a sub-chord, a complete chord must be measured back from J to give prolongation JM.

When driving a curve underground by chord and offset method, it is quite impossible to fix the points on the curve by extending the straight AB and the successive chords forward because the curve has to be driven through solid rock. In such a case, the length of BD backwards to A and the tangent offset FB from A are to be measured. The direction of the first chord BC will then be known and the curve may then be driven to a point just beyond C. Similarly, as the curve proceeds, each chord in turn is projected backwards in order to enable the succeeding offsets to be measured off.

The chord and offset method has the advantage of simplicity but it lacks precision and errors are cumulative in character. The method is suitable for short curves in underground, it may be preferred only due to its simplicity and it can be manipulated by ordinary workmen.

SETTING OUT BY TANGENT AND OFFSETS : This method is only suitable for surface curve where the straights can be lined out to their point of intersection. The curve is set out by rectangular co-ordinates having tangent 'straight' as axis and the tangent point as origin. Alternatly, the curve may be plotted to scale and the offsets measured off at right angle to the two tangents.

A convenient method is to plot the curve to scale and produce the two straights until they intersect at X. Divide TX and XT, equidistantly as B, C and D, and measure their offsets BE, CF and DG. Use these measurements to set out the curve on the ground.



Method of Tangents and offsets

The length of the offsets may also be calcutated as follows :

Let DG = t at a distance TD = L from the tangent point T. Then in triangle POG .

$R^2 = OP^2 + PG^2 = (R - t)^2 + L^2$

$\therefore R^2 - L^2 = (R - t)^2$ and $\sqrt{R^2 + L^2} = R - t$

$\therefore$ Offset $t = R - \sqrt{R^2 - L^2}$

The offsets are set out from both tangets, so minimising the lengths of the offsets required.

This method has the objection that the offsets become very long when nearing the vertex and error can be introduced by faulty alignment or incorrectly setting out the perpendicular. A further disadvantage is that the distances between the points on the curve are not equal.

SETTING OUT BY CHORDS AND ANGLES : The use of rope haulage or locomotive under-ground requires proper setting out of suitable curves to join one road to another. Let it be assumed that an underground haulage roadway AB bearing n 25° E is to be connected to a roadway CD bearing due East with a curve of 45 m radius.

It is now necessary to know **the fundamentals of the curve -**

(i) To find the angle subtended at the centre, BOC = Angle of intersection,
YXC = 90° – 25° = 65°

(ii) To fnd the tangent points B and C,

$XB = XC\ R \tan\frac{65^\circ}{2} = 45 \times \tan 32^\circ 30' = 28.668$ m.

(iii) Let the curve is to be set out by 5 equal chords :

(a) Deflection angle subtended by each chord at the centre of curvature.

$\alpha = \frac{65^\circ}{5} = 13^\circ$

(b) Length of each chord = 2R Sin 6°30' = 2 X 45 X Sin 6°30' = 10.19 m.

(iv) First and last setteng out angles = $\frac{\alpha}{2} = 6^\circ 30'$

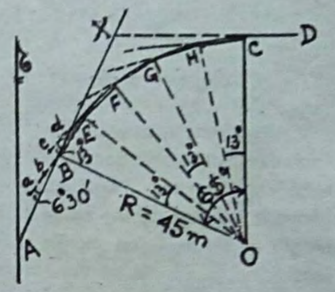
(v) Angle between successive chords = $\alpha = 13^\circ$.

Procedure : A theodolite traverse is made along the roadways to set out base lines near the tangent point B and C, and details such as haulage track, manhole etc. are picked up taking offsets measurements, while measuring the lines on traverse survey.

It is essential to take spot levels at tangent points in order to find any difference in levels, thus making it necessary to drive the curve at a required gradient.

A plan of the proposed curve to a large scale, 1 cm = 1 m, is prepared showing details already picked up.

It is assumed that curve is to be driven from one side only. Station B is esstablished and connected to the base line. The theodolite is set up at B and its sight is aligned along the centre line of the straight BA with the vernier at Zero. The body is clamped and the vernier released. The first setting out angle 6°30' is set out along Bba and the direction is marked by pegs on the roof at b and a or by a white line on the roof. The heading is then driven forward from B in the direction a b B until a full chord length 10.19 m can be measured from B. The position of station E is marked at exactly 10.19 m from B.



Method of Chords and Angles

The instrument is set at E and station B is sighted with the vernier at zero. The body is clamped and the vernier is released. Angle of 13° is turned off from the line EB in the direcction Edc. Two pegs are inserted in the roof along this direction at d and c, and the heading is diverted fromm E in the direction c d E to beyond F. The station F is established at exactly 10.19m from E.

The procedure at E is repeated at other stations to bring the curve to the second tangent point C. Lastly an angle of 6°30' at C will bring the drivage on the direction of the second roadway due East.

SETTING OUT BY RANKINE'S METHOD OR TANGENTIAL ANGLES

In this method the curve is set out by the tangential angles, often called deflecction angles, **with one theodolite and a chain or tape** usually adopted for setting out more important railway curves. The method is simple, quick and accurate and few calculations are necessary. It is based on the principle that equal arcs or chords subtend equal angles at the circumference which are one half the value of the angles subtended by the same arcs or chords at the centre of the circle.

(i) To find the tangential angle A :

The angle TOT_1 is the angle subtended by the chord TT_1 at the centre of the circle and equals 6A and XTT_1 can be regarded as a tangential angle and equals 3A.

$$\text{Angle } TOT_1 = 6A = \frac{\text{arc } TT_1}{2\pi R} \times 360 \text{ degrees}$$

$$\text{Tangential angle} = 3A = \frac{1}{2}\left(\frac{\text{arc } TT_1}{2\pi R} \times 360\right) \text{ degree}$$

$$= \frac{360 \times 60}{4\pi} \times \frac{\text{arc}}{R} \text{ minutes.}$$

$$= 1718.873 \times \frac{\text{arc}}{R} \text{ minutes.}$$

$$\text{ar, Tangential angle} = 1719 \times \frac{\text{chord}}{\text{Radius}} \text{ (in minutes)}$$

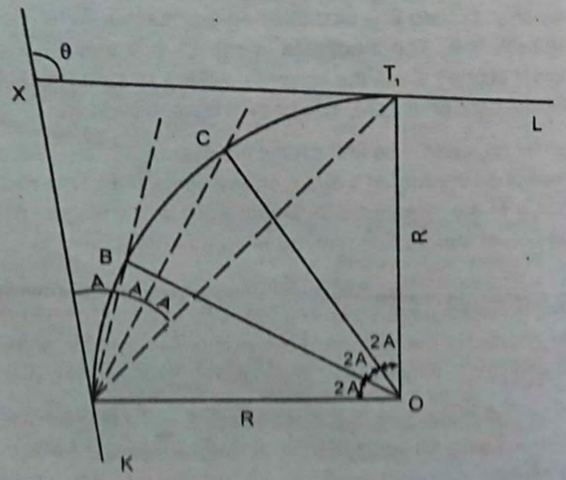
The simplification of the formula is acceptable because, except on very sharp curves, the chord is very nearly the same length as the arc–just slightly less.

(ii) The length of chord

$$= 2R \sin \frac{A}{2}$$

(iii) To locate the tangent points :

Let KT and LT_1 are the two straights whose point of intersection, X, is accessible. The deflection angle is determined and the tangent distance XT is calculated from the formula $R \tan \frac{\theta}{2}$



Method of Tangential angles

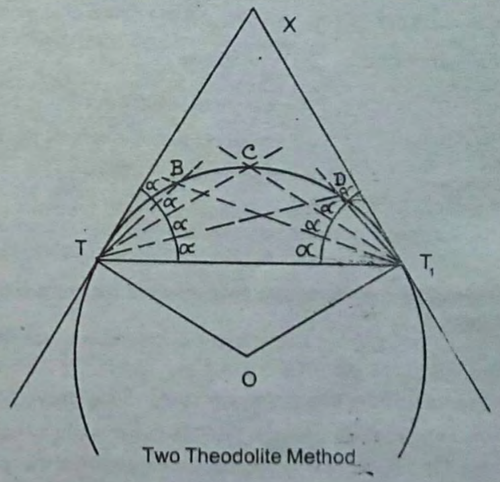
The distance XT is measured back from X along both straights and the tangent from points T and T_1 are accurately located.

Procedure : The theodolite is set up at the tangent point T, the plates are clamped at zero and telescope is directed to the intersection point X and the signal is bisected. The plates are now unclamped and the verniers set to give the first tangential angle A which has been previously calculated. The telescope now points in the direction of the first chord. The length of the first chord TB is measured in that direction and a peg aligned and fixed at B.

The tape or chain is now moved forward, the zero fastened down at B and the assistant prepares to fix point C so that BC equals TB. Simultaneously, the surveyor sets the vernier to read 2A and, when this is done, aligns and fixes point C for the assistant. The operations are repeated for the remaining points on the curve which are equidistantly apart on it.

SETTING OUT BY TWO THEODOLITES : This method, although giving very accurate results, is rarely used because the area on which the curve is to be set out has to be completely unobstructed.

Theodolites are set up at both tangent points-T and T_1. The theodolite at T is set to the first tangent angle α and thus points in the direction of the first point B. Meanwhile, the second theodolite at T_1 has been clamped, with verniers reading zero to bisect the signal at T. The plates are unclamped, the verniers set to the value of the first tangential angle α and the telescope will be pointing in the direction T_1B. The point of intersection of the lines of sight of the two instruments (B) is fixed by an assistant, who moves a surveying pole under the direction of the instrumentmen, until its immage is bisected by the crosswires of both instruments. To determine the second and succeeding points C, D, etc. both instruments are set to the second, third, etc, tangential angles and the corresponding points on the curve are fixed as before.



Two Theodolite Method

The process of fixing a point on the curve until it is bisected by both telescopes is somewhat tedious, but cumulative error is eliminated.

SETTING OUT WHEN INTERSECTION POINT INACCESSIBLE : It frequently happens that the intersection peg cannot be put in owing to an obstacle such as wood, pond, or building, intervening. The following procedure is adopted in setting out the curve -

Select two points C and D on the two straights and between which it is possible to range and measure a straight line CD. Measure angles C and D. Then in triangle CXD,

Angle $XCD = 180^0 - C$

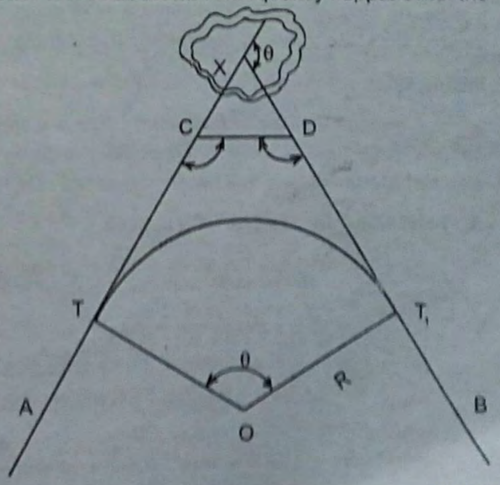
Angle $XDC = 180^0 - D$

$\therefore$ Angle $TXT_1 = 180^0 - (XCD + XDC)$

$= 180^0 - (180^0 - C + 180^0 - D)$

$= C + D - 180^0$

Angle $TOT_1 = \theta = 180^0 - TXT_1$



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TRANSITION CURVES

The purpose of a transition curve is to avoid inconvenience to the passengers, sitting in a vehicle, due to the shock experienced by them when a vehicle suddenly enters on a circular curve from a straight.

Whenever a vehicle travelling along a straight enters a curve, a centrifugal force is produced. This centrifugal force is inversely proportional to the radius of the curve. When the vehicle is moving along a straight, the radius of straight line is infinity. The magnitude of centrifugal force is equal to $\frac{W v^2}{gR}$ where,

W = weight of vehicle in kg

v = speed of vehicle in metres per second

R = Radius of curve in metres

g = acceleration due to gravity = 9.81 m/sec^2

As infinite radius along the straight, suddenly decreases to radius having value equal to R, the centrifugal force suddenly changes from zero to the value $\frac{W v^2}{gR}$

This sudden force produced creates a shock which is source of inconvenience to the passengers. On the other hand if the centrifugal force exceeds a certain value, the vehicle may overturn as the effect of the force is to sway the vehicle outward.

In case of railways, the side thrust is wholly taken by the pressure exerted by the rails on the flanges of the wheels thus causing wear of the rail.

To avoid these effects, hence a small length of curve is needed between the straight and the circular curve. The length should be such that its radius from infinity at the straight should decrease gradually at a certain rate so as to reach to a velue R (the radius of circular curve) when it joins the circular curve. The same type of small curve will also be required when a vehicle moving along the curve is required to join the straight.

Definition :

A transition curve or Easement curve is a curve of varying radius introduced between a straight and a circular curve for the purpose of giving easy changes of direction of a route. It may also be introduced between two branches of a compound curve or reverse curve.

ADVANTAGES OF TRANSITION CURVE

The following are the advantages of providing transition curve at the ends of a circular curve :

(a) It provides comfort to the passengers at the turning without producing any shock.

(b) It allows higher speed at the turnings, the same is not possible otherwise.

(c) It eliminates the danger of derailment and prevents the vehicle from toppling over on curves.

(d) There will be less wear upon the running gear.

FUNCTIONS OF A TRANSITION CURVE

(a) To accomplish gradually the transition from the tangent to the circular curve, and from the circular curve to the tangent, so that the curvature is increased gradually from zero to a specified value and vice versa.

(b) To provide a medium for a gradual increase of **super elevation** from zero on the tangent to the specified value on the main circular curve, so that full amount of super elevation is, attained simultaneously with the curvature of the circular curve at the junction of the transition curve with the circular curve.

CONDITIONS TO BE FULFILLED BY A TRANSITION CURVE

The transition curve should fulfil the following conditions :

(a) It should be tangential to the original straight.

(b) It should meet the circular curve tangentially.

(c) Its curvature should be zero at the origin on straight.

(d) Its radius at the junction with the circular curve should be the same as that of a circular curve.

(e) The rate of increase of curvature along the transition should be the same as that of increase of super elevation.

(f) The length of transition curve should be such that the full superelevation is attained at the junction with the circular curve.

SUPERELEVATION

When a vehicle moves on a curve, there are two forces acting :

(i) Weight of the vehicle and

(ii) The centrifugal force.

Both of these are acting through the centre of gravity of the vehicle. Since the centrifugal force acts horizontally, the effect of which is to push the vehicle off the track. The weight of the vehicle acts vertically. In order to counteract this effect, the plane of the track or road surface is made perpendicular to the resultant of the centrifugal force and the weight of the vehicle. This is attained by raising the outer edge of the track or road above the inner one.

Thus the raising of the outer edge of the road or track over the inner one on curves is called superelevation or cant. The amount of superelevation depends upon the speed of the vehicle and the radius of the curve.

PURPOSE OF SUPERELEVATION

(i) To prevent excessive wear on the outer wheel than the inner wheel.

(ii) To equalize the pressure on the two wheels.

(iii) To prevent the vehicle from over turning.

From mechanics, the centrifugal force is found from formula :

$P = \frac{W v^2}{gR}$, where,

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P = The centrifugal force acting horizontally.

W = The weight of vehicle acting downward.

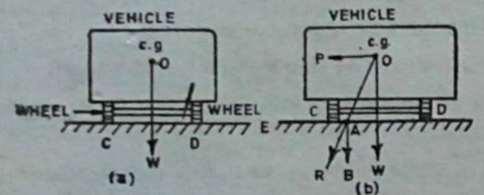
v = The speed of the vehicle in metres per second.

g = The acceleration due to gravity (*i.e., 9.81 m/sec²*)

R = Radius of the curve in metres.

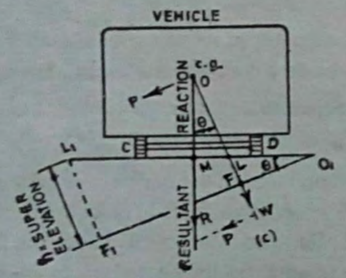
When the vehicle is running along a straight, the only force acting is the weight of the vehicle 'W' acting vertically downward and the weight is equally shared by the two wheels. As soon as the vehicle starts moving on a curve,

there are two forces, one P acting horizontally outward and second W acting downwards. The resultant 'R' of these two forces will be OA, meeting the road surface, at A. Let AE and AB



When the vehicle is running along the straight

Wheh the vehicle is moving on curve without superelevation



When the vehicle is moving on curve with super-elevation provided

are its two components. The horizontal component AE is resisted by the friction between the wheel and ground. The downward component AB is to be shared unequally by the wheels C and D. From Fig. (b) it is clear the load shared by wheel C is more than the wheel D.

Now the position of 'R' depends upon the force P, which in turn depends on the speed of the vehicle. Thus if the resultant R will move nearer and nearer to wheel C with further increase in the force P A time may come when due to increased speed the position of 'R' may pass over wheel C, in which case the whole load will be borne by wheel C and none by wheel D. Thus the vehicle may topple over.

Therefore in order to equalize the pressure on two wheels the outer edge of the road or

track is raised by an amount 'h' called superelevation or cant so that 'R' should be at right angle to surface of road or track.

Derivation of formula for 'h'

Let θ be the angle of inclination of the road or track surface due to raising of the outer edge.

△s OLM and O_1FL are being right angled triangles and similar, because for equilibrium the resultant of weight and the centrifugal force should be equal and opposite to the reaction perpendicular to the road or track surface.

$\therefore \quad \angle MOL = \angle FO_1L = \theta$

Now $\tan\theta = \frac{FL}{O_1F} = \frac{F}{W} = \frac{v^2}{gR}$

$\because$ we have $F = \frac{Wv^2}{gR}$

or $\frac{F}{W} = \frac{v^2}{gR}$

If b = breadth of the road surface or distance between the centre of the rails

then 'h' = $O_1L_1 \sin\theta = b \tan\theta$ (θ being small and $O_1L_1 = b$)

Putting the value of tan θ from above, we have,

$$h = \frac{bv^2}{gR}$$

The amonnt of cant is limited to 15 cm on a standard gauge of 1.5 m.

LENGTH OF TRANSITION CURVE

The length of the transition curve should be such that the required superelevation is provided at a constant rate from the straight to the point of circular curve where it joins.

There are three methods for calculating the length of the transition cvrve.

1. **By an Arbitrary Gradient :**

In this method the super-elevation is provided at a uniform rate, say 1 in n. For super-elevation of 'h' value, the lergth of transition curve will be :

$$L = nh$$

the value of 'n' may vary between 300 to 1200 and

$$h = \frac{bv^2}{gR}$$

2. **By the Time Rate :**

The length of the transition curve may be such that the super-elevation or cant 'h' be applied at an arbitrary time rate of 'a' centimetre per second.

If 'v' is the speed of the vehicle in m/sec, the time taken by the vehicle in passing over length of transition curve

$$= \frac{L}{v} \text{ seconds.}$$

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Thus the superelevation of value 'h' attained in this time

$$= \frac{L}{v} \times a$$

$$\therefore \quad L = \frac{hv}{a} \quad (\textit{The value of 'a' varying from 2.5 to 5 cm})$$

3. By the rate of change of Radial acceleration :

In this method the length of the transition curve is decided so that the passengers may not feel discomfort when the vehicle is passing over the curve.

Mr. Shortt states that the rate of change of radial acceleration of value .3m / sec^3 will be the maximum that the pass unnoticed.

If 'r' is the rate of change of radial acceleration in m / sec^3 Now the time taken to travel over the transition curve

$$= \frac{L}{v} \text{ seconds}$$

∴ Acceleration attained in the time

$$= rt = r \frac{L}{v} \text{ m/sec}^2$$

But radial acceleration on the circular curve

$$= \frac{v^2}{R} \text{ m/sec}^2 \qquad \text{where R = Radius of the curve in metres.}$$

$$\therefore \quad \frac{rL}{v} = \frac{v^2}{R}$$

$$L = \frac{v^3}{rR}$$

$$= \frac{v^3}{0.3R} \text{ m (where r = 0.3m/sec}^3\text{)}$$

Of these methods, the third method is usually used for calculating the length of transition curve.

Centrifugal Ratio

The ratio of the centrifugal force and the weight is called the centrifugal ratio.

∴ Centrifugal ratio $\frac{P}{W} = v^2 / gR$

It is taken as $\frac{1}{4}$ on roads, and $\frac{1}{8}$ for railways

∴ For roads, $\frac{P}{W} = \frac{v^2}{gR} = \frac{1}{4}$ or

$$v = \sqrt{\frac{gR}{4}} \text{ and}$$

For railways $\frac{P}{W} = \frac{v^2}{gR} = \frac{1}{8}$ or $v = \sqrt{\frac{gR}{8}}$

From above expressions with the given speed of the vehicle, minimum radius of the curve can be decided so that the vehicle may pass safely.

Example : *A road deflects at an angle of 60° at a certain point to follow the path of another road. It is desired to connect the two straights with a circular simple curve. If the maximum speed of the vehicle is 60 km/hour and the centrifugal ratio for a road is 1/4, calculate*

(a) The radius of the circular curve, and

(b) The length of the transition curve.

The rate of change of radial acceleration may be taken as 30 cm/sec^3

Solution :

Given v = 60 km/hour $= \frac{60{,}000}{60 \times 60} = \frac{100}{6}$ m/sec

$$\frac{P}{W} = \frac{1}{4} = \text{Centrifugal ratio}$$

$$r = 0.30 \text{ m/sec}^3$$

$$g = 9.81 \text{ m/sec}^2$$

(a) Now we have

$$\frac{P}{W} = \frac{v^2}{gR}$$ Substituting the values, we get

$$\frac{1}{4} = \left(\frac{100}{6}\right) \times \frac{1}{9.81 \times R}$$

or $$R = \left(\frac{100}{6}\right) \times \frac{4}{9.81} = \mathbf{113.3 \text{ metres}}$$

(b) Length of transition curve

$$\text{'L'} = \frac{v^3}{rR} = \left(\frac{100}{6}\right)^3 \times \frac{1}{0.3 \times 113.3}$$

or $$= \mathbf{136.20 \text{ metres}}$$

VERTICAL CURVES

When two different gradients meet, they are connected by a curve in a vertical plane to avoid impact and for manipulating clear visibility. Such a curve is called **Vertical Curve**. It is provided when a highway or a railway crosses a ridge or a valley. In the case of a ridge, a rising gradient is followed by a falling gradient. On the other hand in the case of a valley, a falling gradient is followed by a rising gradient. A vertical curve smoothens the change in gradient so that there is no discomfort or danger to passangers travelling in vehicles.

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The vertical curve may be

(a) A circular arc, or

(b) A parabolic are.

For simplicily in calculation work a circular arc is preferred and is commonly used. The parabolic curve can be easily set out in the field between two different gradients and it provides a good riding comfort as the rate of change in grade is uniform throughout in a parabola having a vertical axis. Thus a parabolic curve contributes to the safety and comfort.

Grade : The gradient or grade of a rail-road or a highway is expressed in two ways

1. *Percentage :* The gradient expressed as a percentage is the vertical rise (or fall) in a horizontal distance of 100m. A gradient of +1% means that the slope rises by 1m in every 100m.

2. *Ratio :* The gradient expressed as a ratio of vertical rise or fall and the horizontal distance. A gradient of 1 in 100 means that there is a rise or fall of 1m in 100m.

Generally, the gradient (g) is expressed as a percentage. A grade is said to be upgrade or +ve grade when elevation along a road or railway increases, while it is said a downgrade or –ve grade when the elevation decreases along the direction of motion.

Rate of change of gradient : The rate (r) of change of gradient is equal to the algebraic difference of the two gradients meeting at the apex divided by the length of curve.

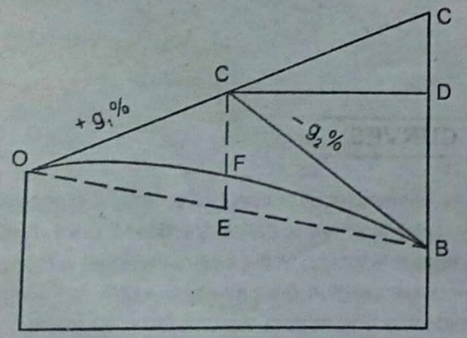
Thus $r = \frac{g_2 - g_1}{r}$ where g_1 = percantage gradient before the intersection point; g_2 = percentage gradient after the intersection point; and L = length of the vertical curve.

The recommended values of the rate of change in gradient for first class railway lines are 0.3% per 100 m for curves at summits and 0.15% for curves at sags. Twice these values are adopted for second class railways or branch lines.

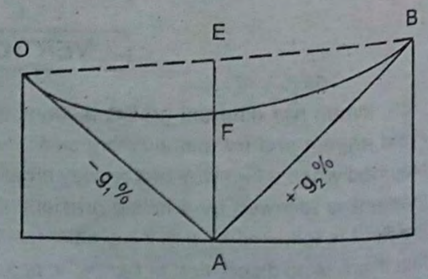
Types of Vertical Curves : Depending upon the magnitude and sign of the gradients at the intersection point, there are six types of vertical curves.

(1) An upgrade ($+g_1\%$) followed by a downgrade ($-g_2\%$). It is a *summit curve* or crest curve.

(2) A downgrade ($-g_1\%$) followed by an upgrade ($+g_2\%$). It is a *valley curve* or sag curve.



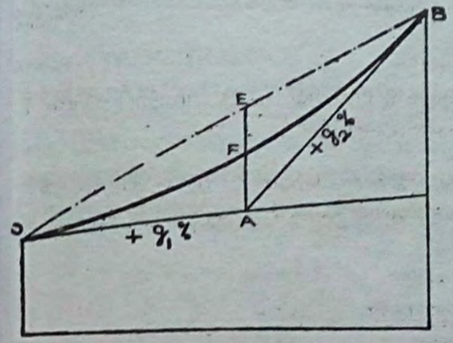
1. Summit or Convex



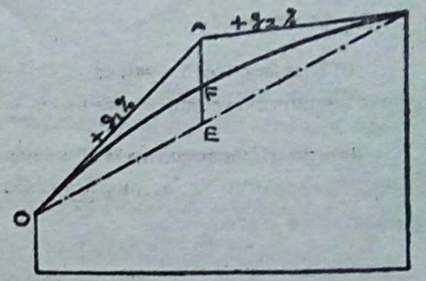
2. Sag or Concave ($g_2 \triangleright g_1$)

(3) An upgrade ($+g_1\%$) followed by another upgrade ($+g_2\%$) where $g_2 > g_1$. It is a sag curve.

(4) An upgrade ($+g_1\%$) followed by another upgrade ($+g_2\%$) where $g_2 < g_1$. It is a summit curve.



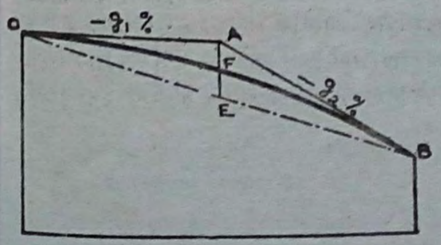
3. Sag or Concave ($g_2 > g_1$)



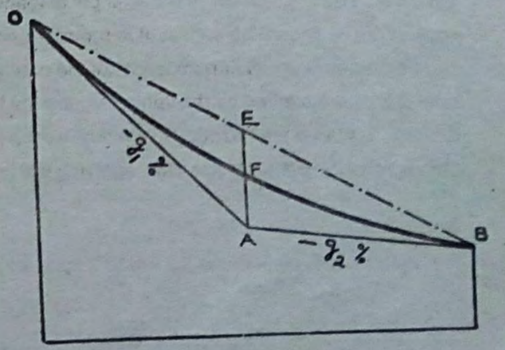
4. Summit or Convex ($g_2 < g_1$)

(5) A downgrade ($-g_1\%$) followed by another downgrade ($-g_2\%$) where $g_2 > g_1$. It is a summit curve.

(6) A downgrade ($-g_1\%$) followed by another downgrade ($-g_2\%$) where $g_2 < g_1$. It is a sag curve.



5. Summit or Convex ($g_2 > g_1$)



6. Sag or Concave ($g_2 < g_1$)

Properties of a Vertical Curve : The following properties of a vertical curve are considered for calculating the various elements of a curve–

(a) A line joining the point of intersection of two tangents and the vertex of a parabola

(vertical curve) when produced, bisects the chord joining the two points.

(b) The vertical curve passes through the middle point of the line joining the intersection point with the middle point of the chord joining the tangent points of the curve.

(c) Off sets from the tangents to the curve are proportional to the square of the distance from the tangent point.

(d) The length of the vertical curve is taken equal to the length of the chord or equal to the sum of lengths of two tangents as the vertical curve is usually flat.

Length of Vertical Curve : The length of vertical curve is obtained by dividing the algebraic difference of the two grades by the rate of change of the grade. Due regard is to be given to the sign of the grade.

$$\text{Length of curve, } l = \frac{\text{Algebraic difference of two grades}}{\text{rate of change of grade}} = \frac{g_1 - g_2}{r_1}$$

Where g_1% (+ve) is one grade and g_2% (+ve) is the other grade. r_1 is the rate of change of grade.

Sight distance on a Vertical Curve : While negotiating a vertical curve, the driver is not able to see the vehicles and other obstacles on the other side of the summit due to curvature of the curve. For the safe driving of the vehicles, a minimum sight distance should be provided on the curve. The sight distance is the length of roadway ahead visible to the driver. The length of the vertical curve should be sufficient to provide the minimum required sight distance. The stopping sight distance is the minimum length of the road which is required for stopping a vehicle. It is the total distance travelled by the vehicle during the following three time periods : (1) the time for the driver to perceive the hazard, (2) the time for the driver to react and apply the brakes, and (3) the time for stopping the vehicle after applying the brakes.

QUESTIONS & ANSWERS

Question 1. What is meant by transition curve ? Why a transition curve is needed on roadways or railways ? What conditions are to be fulfilled by it ? Give the advantages of such a curve.

Answer : TRANSITION CURVES

A transition curve or Easement Curve is a curve of varying radius introduced between a straight and a circular curve for the purpose of giving easy changes of direction of a route. It may also be introduced between two branches of a compound curve or reverse curve.

Necessity : A transition curve is needed on roadways to serve the following functions -

1. To accomplish gradually the transition from the tangent to the circular curve, and from the circular curve to the tangent, so that the curvature is increased gradually from zero to a specified value and vice versa.

2. To provide a medium for a gradual increase of super elevation from zero on the tangent to the specifiaed value on the main circular curve, so that full amount of super-elevation is attained simultaneously with the curvature of the circular curve at the junction of the transition curve with the circular curve.

Conditions to be fulfilled : The transition curve should fulfil the following conditions -

1. It should be tangential to the original straight.
2. It should meet the circular curve tangentially.
3. Its curvature should be zero at the origin on straight.
4. Its radius at the junction with the circular curve should be the same as that of a circular curve.
5. The rate of increase of curvature along the transition should be the same as that of increase of super-elevation.
6. The length of transition curve should be such that the full super-elevation is attained at the junction with the cirular curve.

Advantages : The following are the advantages of providing transition curve at the ends of a circular curve -

1. It provides comfort to the passangers at the turning without poducing any shock.
2. It allows higher speed at the turnings, the same is not possible otherwise.
3. It eliminates the danger of derailment and prevents the vehicle from toppling over on curves.

4. There will be less wear upon the running gear.

Question 2. Write short notes on -

(a) Deviaton curve. (b) Left hand curve and Right hand curve. (c) Vertical curves. (d) Degree of curvature. (e) Super-elevation. (f) Equilibrium cant and cant deficiency.

Answer : SHORT NOTES

(a) **Deviation Curve :** Sometimes on the way of straight path an obstacle (such as bend of river or stream, a building, etc.) is met with, it then becomes necessary to deviate from a given straight path to avoid the intervening obstacle in order to continue along the same straight direction. Thus a deviation curve, which is a combination of two reverse curves having three different or equal radii of three curves, is provided in such situations to overcome the obstacle.

(b) **Left hand & Right hand Curves :** When a curve deflects to the left side of the progress of survey it is called a *left hand curve* and if the curve deflects to the right side of the progress of survey it is known as *right hand curve*.

(c) **Vertical Curves :** When two different gradients meet, they are connected by a curve in a vertical plane to avoid impact and for maintaining clear visibility. Such a curve is known as a *vertical curve*. Vertical curves should be provided for joining two intersecting grade lines of railways and highways in order to round of the angle and to obtain a gradual and smooth change in the grade. Otherwise the vehicles passing over such change in direction or grades will subject to impact that would be either injurious or dangerous.

A vertical curve may be either a *circular arc or a parabolic arc*. The different types of vertical curves are (a) *Summit curve* - when the vertical curve has a convexity upward; and (b) *Valley curve* - when the vertical curve having concavity upwards.

The following **properties of a vertical curve** are considered for calculating the various elements of a curve -

1. A line joining the points of intersection of two tangents and the vertex of a parabola (vertical curve) when produced, bisects the chord joining the two points.

2. The vertical curve passes through the middle point of the line joining the intersection point with the middle point of the chord joining the tangent points of the curve.

3. Offsets from the tangents to the curve are proportional to the square of the distance from the tangent point.

4. The length of the vertical curve is taken equal to the length of the chord or equal to the sum of length of two tangents as the vertical curve is usually flat.

The **length of vertical curve** is obtained by dividing the algebraic difference of the two grades by the rate of change of grade. Due regard is to be given to the sign of the grade.

Length of curve = Algebraic difference of two grades ÷ Rate of change of grade

(d) **Degreee of Curvature :** A curve is designated by its degree of curvature in degrees,

minutes and seconds as in India, the U.S.A.and South America. The degree of curve is the angle subtended at the centre of the curve by a chord of 20 metres. Thus a circular curve is known as a one, two or five degree curve as its 20 metres chord subtends at the centre an angle of 1°, 2° or 5°.

(e) **Super-elevation :** If a vehicle is moving on a curved path on rails or track, the effect of centrifugal force is set up in a horizontal direction perpendicular to the axis of rotation, thus increasing the friction of the wheels on the outer rails and decreasing the friction on the inner rail. If the radius of the curve was small, speed of the vehicle high and the track laid horizontal, there would be a danger of the vehicle become derailed. To avoid the danger the outer rail is raised to a higher level than the inner rail and the amount by which it is raised is called **super elevation or cant.**

$$\text{Super-elevation in cms.} = \text{Track gauge in cms} \times \frac{V^2}{gR}$$

where, V = Speed of vehicle in metre / second

G = 9.81 metre / second^2

R = Radius of the curve in metres.

In no circumstances should the super-elevation exceed 1/8 x track gauge.

(f) **Equilibrium Cant and Cant Deficiency :** When the super-elevation is provided equal to $\frac{bV^2}{gR}$ (where b = breadth of the road surface or distance between the centre of the rails), the load carried by both the wheels will be the same. The springs will be equally compressed and the passengers will not tend to lean in either direction. Such super-elevation is known as the *equilibrium cant.*

On the other hand if it is provided less than this, the outer wheels will carry more load and the outer springs will be more compressed than the inner wheels.The passengers will thus tend to lean outwards. Such a condition is known as *cant deficiency.*

Question 3. Describe the detail how you would lay out a curve 30 m radius to join two roadways at right angles to each other.

Answer : SETTING OUT A CURVE BY CHORDS AND ANGLES

Let AB be the straight underground bearing due North and CD, the second straight, bearing due East. These two roadways meet at X at an angle of 90°. Then

Angle of intersection = (180° - 90°) = 90°.

Angle subtended at the centre = 90° = BOC.

Assuming that the curve is to be set out by 5 equal chords :

(a) Deflection angle subtended by each chord at the centre of curvature = $\frac{90^\circ}{5} = 18^\circ$.

(b) Length of each chord = $2R \sin \frac{18^\circ}{2} = 2 \times 30 \times \sin 9^\circ = 9.39$ metres.

(c) Tangent distance = BX = XC = $R \tan \frac{90^\circ}{2} = 30 \times \tan 45^\circ = 30$ metres.

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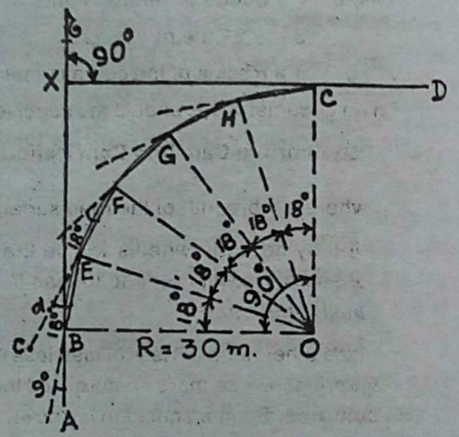
(d) Angle between successive chords = 18°; and first and last setting out angles = 18° ÷ 2 = 9°.

Instruments required : The following instruments are required for the setting out the curve belowground -

1. One theodolite
2. One levelling staff
3. One steel tape
4. Pegs, brass nails, plumb bob, lamps, field-book, pencil, paint, chalk, cord, etc.

Procedure : A theodolite traverse is made along the roadways in which the curve is to be driven and details such as haulage track, manhole etc. are picked up taking offsets while measuring the lines of the traverse survey.

Tangent points B and C are marked by pegs on the roof along the centre line of the roadways AB and DC at a distance of exactly 30 metres from the common point X and spot levels at B and C are taken in order to find any difference in levels between them for laying the curve at required gradient.



A plan of proposed curve to a large scale of 1 cm = 1 m is prepared showing datails already collected.

It is assumed that the curve is to be driven from one side only, from point B and to meet at point C. The theodolite is set up at B. Having made the temporary adjustments its sight is aligned carefully along BX with vernier at zero. The body is clamped and the vernier is released. The first setting out angle 9° is setout and the direction is marked by inserting two pegs into the roof. The roadway is driven from B in the direction BE for a distance of about 11 metres, so that the point E can be located.

The theodolite is again set up at station B and after centering and levelling the station X is correctly bisected with the vernier at zero. The vernier is released and an angle of 9° is set towards BE. A distance of 9.39 metres is accurately measured from station B along the line of sight and the point E is accurately marked with a peg inserted into the roof.

The instrument is next set up at E and the station B is sighted back with the vernier zero. The body is clamped and the vernier is released. An angle of 18° is set off so that the sight is in the direction Edc. Pegs are inserted into the roof at points d and c. The roadway is driven forward from E in the direction cd E until a point beyond E is reached.

The direction of EF is checked with the theodolite at E and the length of EF (9.39 metres) accurately measured to fix the point F.

The same process is repeated at stations F,G and H, and finally the point C is reached. A check on setting operations is made by measuring-(i) the angle HCX with the theodolite at station C which should be exactly 9°,and (ii) the distance of the tangent point C from X should be exactly 30 metres.

The workmen should be instructed -

(i) to extend the centre line in each shift utilising the peg marks left in the roof for the purpose;

(ii) to drive the chord true to centre line;

(iii) to maintain correct width and gradient of the roadway; and

(iv) to inform the surveyor immediately after driving the roadway about 11 metres from the last station mark for checking the accuracy of works already carried out and for setting out the next bearing of the roadway.

Question 4. Explain how an underground road can be set out in simple circular curve.

A road advancing on a bearing N35°12'E is to be turned due north by means of a circular curve of radius 36 metres. Calculate the lengths and quadrant bearings of the chords necessary to set out the curve.

Answer : SETTING OUT CIRCULAR CURVE BELOWGROUND

Simple circular curves are set out belowground by -

(a) Chords and angle method, or

(b) Chords and offset method.

The chords and angle method needs the use of an angular instrument, and a tape, while the chords and offset method does not require any angular instrument.

The chords and offset method has the advantages of simplicity and it can be easily manipulated by ordinary workman in underground, but it lacks precision and errors are cumulative in character. The chord and angle method is more accurate and for laying out more important curves underground this method is adopted the procedure which is as follows :

(i) A theodolite traverse is made along the roadways in which the curve is to be driven and details such as haulage track, manhole etc. are picked up taking offsets while measuring the lines of the traverse survey.

(ii) Tangent points are calculated and located in the underground roadways by means of a wooden plug let into a hole drilled in roof into which is driven a pin.

(iii) Spot levels at the tangent points are taken in order to find any difference in levels between them for laying the curve at required gardient.

(iv) From the known bearings of underground roadways the deflection angle θ is calculated. The curve is then plotted on a large scale 1 cm = 1 metre showing details already collected and the number of chords is decided upon. The angle subtended by each chord at centre of curve is also calculated.

FIELD PROCEDURE :

Having fixed the tangent points T and T_1, angular instrument is set at T. It is sighted along TA and the angle between the chord and straight $\alpha / 2$, where α is the angle subtended by the chord at centre of the curve, is turned. The line of sight would be pointing along TA' along which at least three points could be left in roof so that chord TB' be driven.

When the chord TB has been driven a few metre ahead of B, the instrument would be set at B and sighted along BT. The angle to be turned off would be α as shown in fig. Marks would be left along BB' so that the chord BC could be driven. The same procedure would be followed at C and D and the angle to be turned at these points would be α. The chord driven on D'D should pass through T_1. If T_1 has not been previously established, it should be established, after DT_1 has been driven and angle DT_1P_1 should be checked to equal to $\alpha / 2$.

Appartus Required :

1. Details plan of the curve.
2. Dial or Theodolite.
3. Dumpy levels and levelling staff.
4. Targets for sighting
5. Metallic tape.
6. Wooden plugs, pin, lamps, chalk, paint, chalked cotton thread, field book, pencil etc.

Calculation : Roadways Bearing N 35°12'E to be turned due North by a curve of radius 36 metres :-

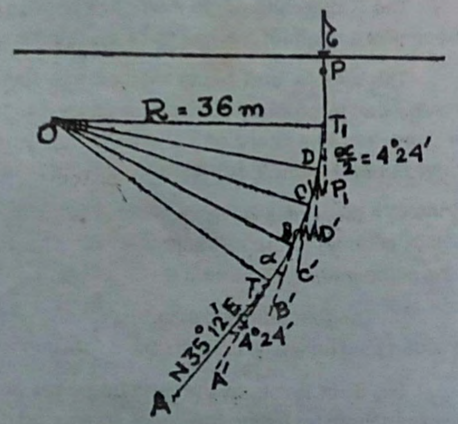
Let AT is the roadway having a bearing of N 35°12'E to be turned on to due north through a curve of radius 36 metres.

(a) Diflection angle = θ = 35°12'

(b) Assuming the curve is to be set out by 4 equal chords of equal length. Therefore angle subtended at O by each chord = α = 35°12' ÷ 4 = 8°48'

(c) Angle between successive chords = α = 8°41'

(d) Angle between straight and chord = first and last setting out angles = $\alpha \div 2$ = 4°24'



Setting Out a Curve

(e) Length of each chord = 2R sin 4°24' = 5.523m.

(f) Tangent distance =R tan 35°12' / 2 = 36 x tan 17°36'm = 11.42 m.

Details of Chords –

Chord	Calculation	Quadrant Bearing	Length of Chord (mts)
	AT = N 35°12'E		
TB	N 35°12'E - 4°24'	N 30°48'E	18.41
BC	N 30°48'E - 8°48'	N 22°00'E	18.41
CD	N 22°00'E - 8°48'	N 13°12'E	1841
DT_1	N 13°12'E - 8°48'	N 4°24'E	18.41
	T_1H = N4°24'E - 4°24'	Due North (bearing of straight checked)	

WORKED OUT EXAMPLES

Example 1. Find the radius of a 2° curve with a 20 metres chord.

Answer : We know, $R \sin \frac{1}{2} D^0 = \frac{\text{Chord}}{2}$

When, R = Radius of curve

D = Degree of curvature.

$\therefore$ $R = \text{Chord} / 2 \div \sin \frac{1}{2} D^0 = \frac{20}{2} \div \sin \left(\frac{1}{2} \times 2^0\right)$

$= 10 \div \sin 1^0 =$ **572.986 metres.**

Example 2. What is the degree of curvature of a 200 m curve ?

Answer : Let the degree of curvature being the angle subtended at the centre by chord of 20 metres. Now –

$\text{Chord} = 2R \sin \frac{\theta}{2}$ and thus $20 = 2 \times 200 \sin \frac{\theta}{2}$

Hence $\sin \frac{\theta}{2} = 0.05$; $\frac{\theta}{2} = 2°51'58"$

$\therefore$ Degree of curvature, θ = 5°43'56"

Example 3. Two roadways in a coal seam are to be connected by a circular curve. The intersection angle is 104°24' and the radius of the curve is 75 metres. Calculate –

(a) The tangent distance; (b) The length of the curve; (c) The length of each chords if the chords are to be 10 equal chords; (d) The deflection angle of the first chord and the deflection angle for subsequent chords; (e) The rise of the curve; (f) The length of the main chord.

Answer : Tangent distance = $R \tan \frac{\theta}{2}$

$= 75 \tan \frac{104°24'}{2} = 75 \tan 52°12' =$ **96.69 metres.**

(b) **Length of curve** = $2\pi R \frac{\theta}{360°}$

$2 \times \frac{22}{7} \times 75 \times \frac{104°24'}{360°} =$ **136.714 metres.**

(c) As there are 10 equal chords, each small chord subtends an angle α at the centre.

$\alpha = \frac{104°24'}{10} = 10°26'24"$

Length of chord = $2R \sin \frac{\alpha}{2} = 2 \times 75 \sin 5°13'12"$

= 13.647 metres.

(d) **Deflection angle** = tangential angle = $\sin^{-1} \frac{\text{chord}}{2R}$

$= \sin^{-1} \frac{13.647}{2 \times 75} = 5°13'12"$

(e) **Rise of curve** = $R\left(1 - \cos \frac{\theta}{2}\right) = 75(1 - \cos 52°12') =$ **29.03 metres.**

(f) **Main chord** = $2R \sin \frac{\theta}{2} = 2 \times 75 \times \sin 52°12' =$ **118.523 metres.**

Example 4. A left-hand circular curve of 30 m radius is set out between two underground roadways meeting at 120°. If the curve is set out by 5 equal chords, calculate –

(a) the length of each chord,

(b) the tangent distance, and

(c) the first and last setting out angles.

Answer : Angle at the centre = $180° - 120° = 60°$

As there are 5 equal chords, therefore each chord subtending an angle of $60° \div 5 = 12°$ at the centre.

(a) **Length of each chord** = $2R \sin \frac{\theta}{2} = 2 \times 30 \times \sin \frac{12°}{2}$

$= 2 \times 30 \times \sin 6° =$ **6.272 metres.**

(b) **Tangent distance** = $R \tan \frac{60°}{2} = 30 \times \tan 30° =$ **17.320 metres.**

(c) First and last setting out angles = $\frac{12°}{2} = 6°$

Example 5. Two roads AB and AC have bearings of S57°10'E and S47°30'W. They are to be connected by a curve tangential to both roads, with a radius of 75 metres and set out from the tangent point on AB. Calculate (a) the intersection angle, (b) the length of the curve, (c) the length of the chords if there are to be 15 equal chord, (d) the bearings of the first two chords, (e) the tangent distance.

Answer : (a) **Intersection of angle** = $180° - (57°10' + 47°30') =$ **76°20'**

(b) **Length of Curve** = $2\pi R \times \frac{76°20'}{360°}$

$= 2 \times \frac{22}{7} \times 75 \times 0.21203 =$ **99.94 metres.**

(c) **Length of chord** = $\frac{98.65}{15} =$ **6.66 metres.**

(d) **Deflection angle** = $\frac{6.66}{75} \times 1719$ minutes = **2°32'35"**

Bearing of the first Chord = N (57°10' + 2°32'35")W

= N 59°42'35"W

Bearing of the Second Chord = N 59°42'35"W + 2 x 2°32'35"

= N 64°47'45"W

(e) **Tangent distance** = 75 tan 75°20'/2 = **57.90 metres.**

Example 6. The angle of deflection from one straight portion of a railway to another is 40°. The straights are to be connected by a curve having a radius of 10 chains. Calculate –

(a) Tangent distances

(b) Main chord

(c) The rise

(d) Length of curve

(e) Tangential angle corresponding to first chord of 1 chain long.

Answer : Given Radius R = 10 chains, and $\theta = 40°$

(a) **Tangent distances** = $R \tan \frac{\theta}{2} = 10 \times \tan \frac{40°}{2} =$ **3.64 chains**

(b) **Main chord** = $2R \sin \frac{\theta}{2} = 2 \times 10 \times \sin 20° =$ **6.84 chains**

(c) **The rise** = $R\left(1 - \cos \frac{\theta}{2}\right) = 10 (1 - \cos 20°) =$ **0.603 chains**

(d) **Length of curve** = $2\pi R \frac{\theta}{360°} = 2 \times \frac{22}{7} \times 10 \times \frac{40°}{360°} =$ **6.98 chains**

(e) **Tangential angle** = $\sin^{-1} \frac{C}{2R} = \sin^{-1} \frac{1}{2 \times 10} = 2°52'$.

Example 7. Two roads AB and BC intersect at B and the deflection angle is 105°. These roads are to be connected with a circular curve 10 chains radius. Determine :

(a) Tangent distances, (b) Main chord, (c) Distances from B to centre of curve, (d) Length of curve, and (e) Number od chords of 1 chain length.

Answer : (a) Tangent distance $= R \tan \frac{\theta}{2}$

$= 10 \times \tan \frac{105^\circ}{2}$ chains

$= 13.032$ chains.

(b) Main chord $= 2R \sin \frac{\theta}{2} = 2 \times 10 \times \sin \frac{105^\circ}{2} = 15.867$ chain

(c) Distance of B from centre of curve

$= \frac{R}{\cos \frac{\theta}{2}} = \frac{10}{\cos 52^\circ 30'} = 16.427$ chains

(d) Length of the curve $= 2\pi R \frac{\theta}{360^\circ} = 2 \times \frac{22}{7} \times 10 \times \frac{105}{360}$

$= 18.334$ chains.

(e) Number of chords of one chain length

$= \frac{\text{Length of curve}}{\text{Length of chord}} = \frac{18.334}{1} = 18.334$

= 18 nos. full chain + 33.4 link .....[1 ch = 100 links]

Example 8. The co-ordinates, in metres of three stations A, B and C are as follows :

Station	Latitude	Departure
A	N 30	E 40
B	S 60	E 70
C	S 20	W 20

Calculate the radius of a curve to pass through the three stations.

Answer : To find out radius of a Curve.

$AB = \sqrt{(70 - 40)^2 + (80 + 60)^2} = 143.2$ m

$BC = \sqrt{(60 - 20)^2 + (70 + 20)^2} = 68.49$ m

$CA = \sqrt{(20 + 80)^2 + (20 + 40)^2} = 116.6$ m

We know from the sine rule in a triangle ABC.

$\frac{AB}{\sin C} = \frac{BC}{\sin A} = \frac{AC}{\sin B} = 2R$ where R is radius of the circumscribed circle.

Knowing the co-ordinates of the terminal stations of a line, the tangent of the bearing angle = difference in departures ÷ difference in latitudes.

Bearing of AB $= \tan^{-1} \frac{70 - 40}{(-60) - (80)} =$ S 12°06'E

Bearing of AC $= \tan^{-1} \frac{(-20) - (40)}{(-20) - (80)} =$ S 30°58'W

Angle BAC = 12°06' + 30°58' = 43°04'

$\frac{BC}{\sin BAC} = 2R \ldots R = \frac{BC}{2 \sin BAC} = \frac{98.49}{2 \sin 43^\circ 04'} = 72.12$m

Therefore **the radius of the circular curve** passing through A, B, and C is **72.12m.**

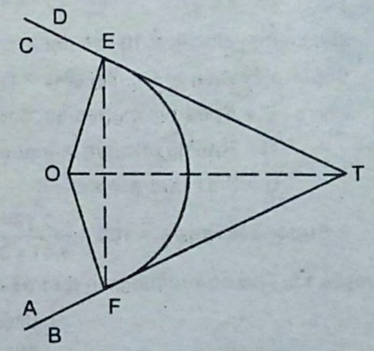
Example 9. Two roadways AB and CD are to be joined by a curved road 60m radius. The bearings of the roads are AB = 37°10' and CD = 110°04'.

AB and CD meet at T when extended. The co-ordinates of B, D and T in metres are as follows :

	X	Y
B	2130.72	73.20
D	2114.81	222.80
T	2205.80	172.23

How far must the roads be extended to meet the tangent points ? What will be the length of curved road from B to D ?

Answer : Let AB and CD be the two roadways and E and F be the tangent points for the curve having a radius = 60 metres.



Bearing of AB = 37°10'

Bearing of CD = 119°04'

∠DTB = 119°04' - 37°10' = 81°54'

Tangent distance ET = FT

$= R \cot 40^\circ 57'$

$= 60 \cot 40^\circ 57' - 69.1438$m

Length of curved road EF

$= 60 \times 2\pi \times \frac{180^\circ - 81^\circ 54'}{360^\circ}$

$= 60 \times 2 \times \frac{22}{7} \times \frac{98^\circ 06'}{360^\circ}$

= 102.7714 metres.

Also, BT $= \sqrt{2205.80 - 2130.72)^2 + (172.23 - 73.20)^2}$

$= \sqrt{5637.006 + 9806.941} = 124.274$ metres.

BF = BT – FT = 124.274 – 69.1438 = 55.130 metres

Also, DT $= \sqrt{(2205.80 - 2114.81)^2 + (172.23 - 222.80)^2}$

$= \sqrt{827.180 + 2557.325} = 104.098$ metres.

Now, DE = DT – ET = 104.098 – 69.1438 = 34.954 m.

The roadways AB and CD must be driven 55.130m and 34.954m respectively to meet the tangent points. **Length of curved road EF is 102.77 metres.**

Length of roadway DEFB = 55.130 + 102.77 + 34.954 = **192.854 metres.**

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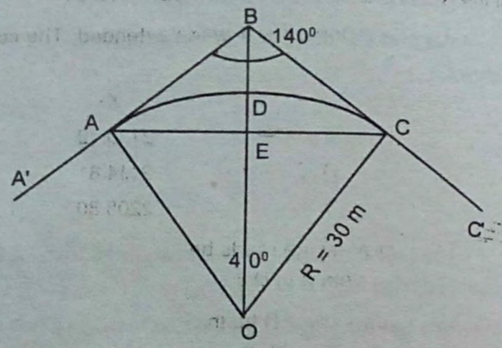
Example 10. Two straight portions of underground roadway include an angle of 140 degrees and they are to be joined by a circular curve having a radius of 30 metres. The gauge of track is 1.05 metres, and the speed of train going round the curve is not to exceed 10 Km per hour. Find the tangent distances, the rise of curve, the length of curve and super-elevation of the outer rail.

Answer : Let A'A and C'C represent the straight roadways which include an angle ABC of 140°

Radius of curve = 30 metres. Angle at the centre = AOC = 180° – 140° = 40°.



Tangent distance = AB = BC = AO. cot ABO = 30 cot 70° = 30 x 0.36397 = **10.92 m**

$$\text{Length of curve} = 2\pi R \times \frac{\angle AOC}{360^0}$$

$$= 2 \times \frac{22}{7} \times 30 \times \frac{40^0}{360^0}$$

∴ **Length of curve = 20.952 m.**

Rise of the curve = DE

$$= R\left(1 - \text{Cos}\ \frac{AOC}{2}\right) = 30(1 - \cos 20^0) = 30(1 - 0.93969) = \textbf{1.809 metres.}$$

Maximum velocity = 10 Km per hour = 2.78 m per sec.

Super-elevation or Cant in cms = Track gauge in cms x $\frac{V^2}{gR}$

where V = Speed in metres/second

R = Radius of curve in metres.

g = 9.81 metres/sec²

∴ **Super-elevation** = $105 \times \frac{2.78^2}{9.81 \times 30}$ = **2.76 cms.**

Example 11. The co-ordinates in metres of three points A, B and C are :

	Latitude	Departure
A	- 491.2	+ 161.3
B	+188.4	+ 337.6
C	+ 296.7	- 212.9

Calculate the radius of the circular curve passing through A, B and C. Verify the results by plotting to a suitable scale.

Answer : We know from the sine rule, in a triangle ABC,

$\frac{AB}{\text{Sin C}} = \frac{BC}{\text{Sin A}} = \frac{AC}{\text{Sin B}} = 2R$, where R is the radius of the circumscribed circle.

Knowing the co-ordinates of the terminal stations of a line, the tangent of the bearing angle = Diff. in Dep. ÷ Diff. in Lat.

$$\text{Bearing of AB} = \tan^{-1}\frac{(337.6) - (161.3)}{(188.4) - (-491.3)} = \tan^{-1}\frac{176.3}{679.6} = \text{N } 14^0 33'\text{E}$$

$$\text{Bearing of BC} = \tan^{-1}\frac{(-212.9) - (337.6)}{(296.7) - (188.4)} = \tan^{-1}\frac{-550.5}{108.3} = \text{N } 78^0 52'\text{W}$$

$$\text{Bearing of AC} = \tan^{-1}\frac{(-212.9) - (161.3)}{(296.7) - (-491.2)} = \tan^{-1}\frac{-374.2}{787.9} = \text{N}25^0 24'\text{W}$$

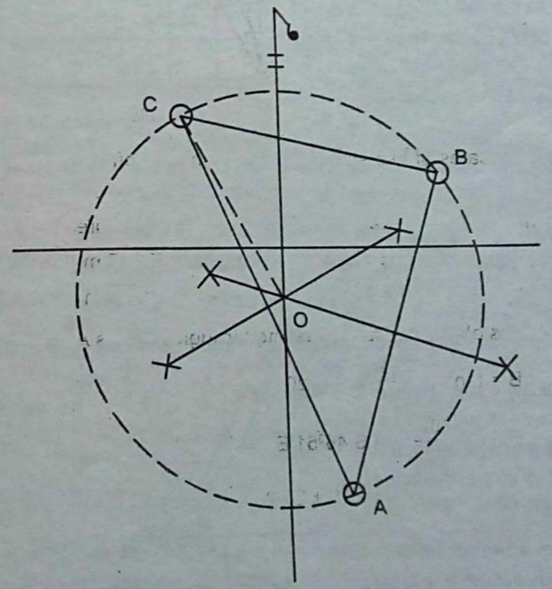
In triangle ABC,

Angle CBA = 25°24' + 14°33' = 39°57'

$$\text{Length of BC} = \frac{\text{Difference in dep.}}{\text{Sine bearing angle}} = \frac{550.5}{\text{Sin } 78^0 52'} = 561.1 \text{ m}$$

$$\text{Now, } \frac{BC}{\text{Sin A}} = \frac{561.1}{\text{Sin } 39^0 57'} = 2R$$

$$\text{Therefore, } R = \frac{561.1}{2\text{Sin } 39^0 57'} = 436.2 \text{ metres.}$$



Scale : 1 cm = 100 m

Hence, the **radius of the curve** passing through A, B and C are 436.9 metres.

PLOTTING : Positions of points A, B and C are plotted from their given co-ordinates to the scale 1 cm = 100 metres.

The points A, B and C are joined to form the triangle ABC. Perpendicular bisectors of AB and AC are meeting at 'O' which is the centre of the circle. OC joined. Then radius of the circle = OC which is measured. Measured distance of OC = 437.5 metres.

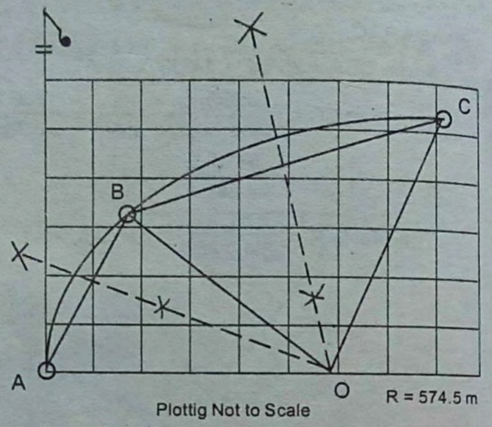
Example 12. Co-ordinates of two points B and C in relation to a point A are as follows.

B	380 m North	142 m East
C	520 m North	824 m East

Find out by plotting the radius of the curve passing through three points A, B and C.

Answer : The positions of points A, B and C are plotted from their given co-ordinates to scale 1 cm=100 m.

The points, A, B and C are joined to form a triangle ABC. Perpendicular bisector of AB and BC are meeting at O which is the centre of the circle. Then the radius of the circle = OC =OA =OB. The measured distancae of the radius, R = 574.5 metres.



Plottig Not to Scale

Example 13. The co-ordinates of two stations B and C in relation to another station A are the following -

Station	Latitude	Departure
B	N 576 m	E 215 m
C	S 750 m	E 800 m

Calculate the radius of circular curve passing through stations A, B and C.

Answer : Bearing of AB = $\tan^{-1} N\frac{215}{576}$ E = N 20°28'E

Bearing of AC = $\tan^{-1} S\frac{800}{750}$ E = S 46°51'E

Therefore, angle BAC = 180° – (46°51' + 20°28') = 112°41'

Length of BC = $\sqrt{(576 + 750)^2 + (800 - 215)^2}$ = 1449.3 metres

We know, $\frac{a}{\text{Sin A}} = 2R$, therefore, $R = \frac{a}{2 \times \text{Sin A}} = \frac{a}{2 \times \text{Sin } 112°41'}$

$= \frac{1449.3}{2 \times \text{Sin } 112°41'} = 785.40$ metres

The radius of a circular curve passing through A, B and C = **785.40 metres.**

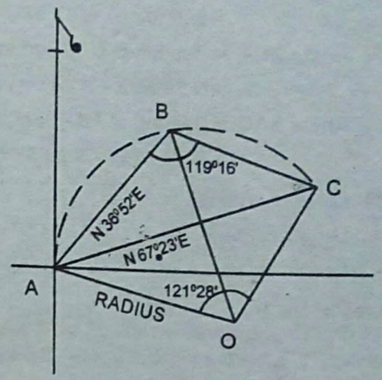
Example 14. A curve passes through three points having the following co-ordinates in metres :

Points	Latitude	Departure
A	00	00
B	60 N	45 E
C	50 . .	120 E

Calculate the radius of the curve.

Answer : Let O is the centre of the curve. Then OA = OB = OC = the radius of the curve.

AC is the long chord. The angle AOC subtended at the centre by the long chord AC is equal to 360°-twice the angle ABC contain between AB and BC.



Bearing of AB = N $\tan^{-1}\frac{45}{60}$ E

= N 36°52'E

Bearing of AB = N $\tan^{-1}\frac{120}{50}$ E

= N 67°23'E

Co-ordinates of C relative to B = 10 m South and 75 m East.

Bearing of BC = S $\tan^{-1}\frac{75}{10}$ E = S 82°24'20"E or Azimuth = 97°35'40"

Angle ABC = Azimuth BA – Azimuth BC = 216°52' – 97°35'40" = 119°16'20"

Angle AOC = 360° – 2 x 119°16'20" = 360° – 238°32'40" = 121°27'20"

Half angle AOC = 121°27'20" ÷ 2 = 60°43'40"

Length of chord AC = $\sqrt{20^2 + 120^2}$ = 130 metrs

Radius of curve = Chord $\frac{\text{AC}}{2\text{Sin}\frac{\text{AOC}}{2}} = \frac{130}{2 \times \text{Sin } 60°43'40"}$ = **74.510 m**

Example 15. The co-ordinates of three points A, B and C are as follows :

A	769m N	823m E
B	447m N	674m W
C	835m S	100m W

Calculate the angles and sides of the triangle ABC, the area of the triangle and radius of the circle which passes through vertices of the figure. Check your answer by plotting.

Answer : Calculation of area of a triangle and the radius of the circle passing through vertices of the figure -

Bearing of line AB = $\tan^{-1}\frac{-674 - 823}{447 - 769} = \tan^{-1}\frac{+574}{-1282}$ = S 77°52'W

Bearing of line BC = $\tan^{-1}\frac{-100 + 674}{-835 - 447} = \tan^{-1}\frac{+574}{-1282}$ = S 24°07'W

Bearing of line CA = $\tan^{-1}\frac{823 + 100}{769 + 835} = \tan^{-1}\frac{923}{1604}$ = N 29°55'E

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1. (a) **Angle ABC** = Bearing of BC - Bearing of BA
= S 24°07'E - N 77°52'E = 155°53' - 77°52' = 78°07'

(b) **Angle BCA** = Bearing of CA - Bearing of CB
= N 29°55'E - N 24°07'W = 54°02'

(c) **Angle BAC** = Bearing of AB - Bearing of AC
= S 77°52'W - S 29°55'W = 47°57'.

2. (a) Length of line **AB** $= \sqrt{(447 - 769)^2 + (-674 - 823)^2} = 1531.239$ m

Length of line **BC** $= \sqrt{(-835 - 447)^2 + (100 - 674)^2} = 1404.635$ m

Length of line **CA** $= \sqrt{\{769 - (-835)\}^2 + \{-100 - 823\}^2} = 1850.606$ m.

3. **Area of the triangle ABC**

$$= \frac{1}{2} \times a \times b \times \text{Sin } C = \frac{1}{2} BC \times AC \times \text{Sin } BCA$$

$$= \frac{1}{2} \times 1404.635 \times 1850.606 \times \text{Sin } 54°02' = \mathbf{105193.2 \text{ Sq. m.}}$$

4. **Radius of circle**

$$2R = \frac{BC}{\text{Sin } BAC} = \frac{1404.635}{\text{Sin } 47°57'} = \mathbf{1891.6099 \text{ metres.}}$$

$\therefore$ Radius = R = 1891.6099 ÷ 2 = **945.805 metres.**

Example 16. The co-ordinates in metres of two points B and C with respect to A are :

B	500 N	470 E
C	550 N	770 E

Calculate the radius of a circular curve passing through the three points and the co-ordinates of the tinter-section point I, assuming A and C are tangent points.

Answer : Bearing of AB $= \tan^{-1} N \frac{470}{500} E = N\ 43°13'42"E$

Bearing of AC $= \tan^{-1} N \frac{770}{550} E = N\ 54°27'44"E$

Therefore, the angle BAC = 54°27'44" – 43°13'42" = 11°14'02"

Length of BC $= \sqrt{(550 - 500)^2 + (770 - 470)^2} = 304.138$ m

We know, $\frac{a}{\text{Sin } A} = 2R$

$\therefore$ Radius = R $= \frac{a}{2 \text{ Sin } A} = \frac{304.138}{2 \times \text{Sin } 11°14'02"} = 780.583$ m

Length of long chord = AC $= \sqrt{550 + 770^2} = 946.24$ m

Again, $\text{Sin } \frac{\theta}{2} = \frac{\text{chord}}{2R} = \frac{946.24}{2 \times 780.58358} = 0.6061106$

When $\frac{\theta}{2} = 37°18'30"$ $\therefore \theta = 74°37"$

θ = Deflection angle = Angle subtended at centre.

Angle IAC $= \frac{\theta}{2}$

Bearing of AI = (54°27'44" - 37°18'30") NE = N 17°09'14"E

Length of AI = Tangent distance = R tan $\frac{\theta}{2}$

= 780.58358 x tan 37°18'30" = 594.82 m

Co-ordinates of Intersection point I :

Latitude = 594.82 Cos 17°09'14" N = 568.36 m(N)

Departure = 594.82 Sin 17°09'14"E = 175.43 m(E)

Example 17. Two tangents intersect at chainage 1190 m. The deflection angle being 36°. Calculate all the data necessary for setting out a curve with a radius of 300 m, if the peg intervals are 30 m.

Answer : Deflection angle 36°; Apex angle = 190° - 36° = 144°; Radius of curve = 300 m.

Tangent length (T) = R tan $\frac{36°}{2} = 97.47$ m

Length of curve = $\pi R \frac{36}{180} = \pi \times 300 \times \frac{36}{180} = 188.52$ m

Chainage of the intersection point = 1190.00 m.

Deduct tangent length (T) = – 97.47 m.

Chainage of the 1st tangent point = 1092.53 m.

Add length of the curve = +188.52 m.

Chainage of the 2nd tangent point = 1281.05 m.

The curve should consist of seven chords : five chords and two sub-chords.

Length of the first sub-chord = 1100 - 1092.53 = 7.47 m

Length of the unit chords (Second to Sixth) = 30 m

Length of the last chord = 31.05 m

The last chord is chosen slightly greater than the unit chord as otherwise the last chord would have a length of 1.05 m which is not convenient.

By Deflection Angles : The tan tangential angles are -

for the first chord = 1718.9 x $\frac{7.47}{300}$ minutes = 42'48"

for the second to sixth chords = 1718.9 x $\frac{30}{300}$ min = 2°51'53"

for the last chord = 1718.9 x $\frac{31.05}{300}$ minutes = 2°57'50"

The total tangential angles for the chords are -

First chord		= 42'48"
Second chord	= 42'48" + 2°51'53"	= 3°34'41"
Third chord	= 3°34'41" + 2°51'53"	= 6°26'34"
Fourth chord	= 6°26'34" + 2°51'53"	= 9°18'27"
Fifth chord	= 9°18'27" + 2°51'53"	= 12°10'17"
Sixth chord	= 12°10'17" + 2°51'53"	= 15°02'10"
Seventh chord	= 15°02'10" + 2°57'50"	= 18°00'00"

Check = Total tangential angles for the seventh chord = $\frac{1}{2}$ x Deflection angle = $\frac{1}{2}$ x 36° = 18°.

Example 18. Two straight underground roads being driven towards a point P are to be connected by circular curve of 200 m radius and the following are notes of a traverse made to connect them :

	Line	Bearing	Distance
Straight	AB	S 19°22'W	100 m
	BC	S 87°42'E	230 m
	CD	N 59°54'E	306 m
	DE	N 03°18'W	394 m
Straight	EF	S 84°16'W	100 m

Calculate the distance AT_1 and FT_2 where T_1 and T_2 are the tangent points on the curve.

Answer : The traverse is roughtly plotted. It is, first of all, necessary to calculate the length and bearing of the closing line AF and also to find the lengths AP and FP.

Line/Stn.	Quadrant Bearing	Distance mts.	Partial Co-ordinates		Total Co-ordinates	
			Lat	Dep	Lat	Dep
A					0	0
AB	S 19°22'W	100				
B			– 94.34	– 33.16	– 94.34	– 33.16
BC	S 87°42'E	230				
C			– 9.23	+ 229.81	– 103.57	+ 196.65
CD	N 59°54'E	306				
D			+ 153.46	– 263.74	+ 49.89	+ 461.39
DE	N 03°18'W	394				
E			+ 393.35	– 22.68	+ 443.24	+ 438.71
EF	S 84°16'W	100				
F			– 9.99	– 99.50	+ 433.25	+ 339.21

Bearing of AF = $\tan^{-1}\frac{339.21}{433.25}$ = N 38°04'E

Length of AF = $\sqrt{339.21^2 + 433.25^2}$ = 550.24 m

In triangle APF :

= 38°04' – 19°22' = 18°42'

Angle AFP

= 84°16' – 38°04' = 46°12'

Angle APF

= 180° – (84°16' – 19°22') = 115°06'

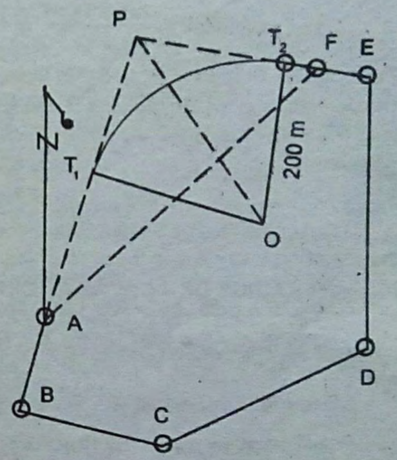
Total = 180°00'

$\frac{PF}{\text{Sin } 18°42'} = \frac{550.24}{\text{Sin } 115°06'}$

∴ PF = 194.81 metres.

$\frac{PA}{\text{Sin } 46°11'} = \frac{550.24}{\text{Sin } 115°06'}$

∴ PA = 438.56 metres.



Again, the angle T_1PT_2 – 115°06' = Apex angle.

∴ Intersection angle = Angle at centre made by the curve = θ = 180° - 115°06' = 64°54'

Tangent distances = $PT_1 = PT_2 = R \tan\frac{\theta}{2}$

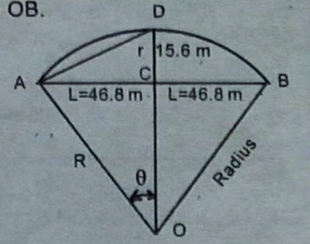
= 200 x tan 32°27' = 127.17 m.

Therefore, **length of FT_2** = 194.81 – 127.17 = 67.64 m

and **length of AT_1** = 438.56 – 127.17 = 311.39 m

Example 19. In making survey of a colliery rail yard it is found that the base line intersects the inner or concaved side of a curved rail at two points. A and B, 93.6 m apart, and the offset CD midway between these two points to the side of the rail is 15.6 m. Calculate the radius of the curve and the length of the rail between the two points intersected by the base line.

Answer : In the adjoining sketch, let r be the rise CD = 15.6 metres, tet L = the half-chord AC = (93.6 ÷ 2) = 46.8 metres. Let R = the radius = OA = OD = OB.



Radius of the Curve :

In triangle AOC, $R^2 = OC^2 + AC^2 = (R - r)^2 + L^2$

$= R^2 - 2Rr + r^2 + L^2$

Hence $2Rr = r^2 + L^2$

Substituting known valus, 2R x 15.6

$= (15.6)^2 + (46.8)^2$

$\therefore$ **Radius = R** $= \frac{(15.6)^2 + (46.8)^2}{2 \times 15.6}$ **= 78 metres**

Length of Curve ADB;

$\text{Sin}\theta = \frac{AC}{OA} = \frac{46.8}{78} = 0.6 \quad \therefore \theta = 36^0 52'$ and $\angle AOB = 73^0 44'$

$\therefore$ Length of Rail $= \frac{2\pi R \times 74^0 44'}{360^0} = 100.42$ metres.

Example 20. Station A, B, and C in the workings of a seam are to be joined by a circular curve. The co-ordinates of the stations B and C in relation to A are-

Station	Latitude	Departure
B	N 576 m	E 215 m
C	S 750 m	E 800 m

Calculate the radius of a circular curve to pass through the stations A, B and C. Calculate the area of the circle and the area of the triangle ABC. Estimate the ratio of these two areas.

Answer : RADIUS OF A CIRCULAR CURVE

Bearing of AB $= \tan^{-1} N\frac{215}{576} E = N\ 22^0 28'E$

Bearing of AC $= \tan^{-1} S\ \frac{800}{750} E = S\ 46^0 51'E$

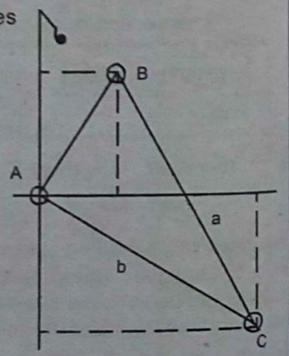
Therefore, angle BAC $= 180^0 - (46^0 51' + 20^0 28') = 112^0 41'$

Length of BC $= \sqrt{(576 + 750)^2 + (800 - 215)^2} = 1449.3$ metres

We know, $\frac{a}{\text{Sin } A} = 2R;$

Therefore, $R = \frac{a}{2 \times \text{Sin } A}$

$= \frac{1449.3}{2 \times \text{Sin } 112^0 41'} = 785.40$ m.



The radius of a circular curve passing through A, B and C = 785.40 metres.

Area of the Circle :

Radius of circle = 785.40 m

Area of the circle $= \pi R^2 = \pi \times (785.40)^2 = 1938681.4$ Sq.m.

Area of triangle ABC

Stn.	Total Lat	Total Dep	Sum of Lats	Difference of Deps	Double Areas Sq. m.	
A	0	0	+576	–215	-	–123840
B	+576	+215	–174	–585	+101790	-
C	–750	+800	–750	+800		–600000
A	0	0				

+101790 -723840

+101

Double Area = 6220

Area = 622050 ÷ 2 = 311025 Sq. metres.

Ratio of the circle and triangle ABC

Area of circle : Area of triangle

1938681.4 : 311025

Ratio = 6.23 : 1

PROBLEMS

Problem 1. It was proposed to connect two straights, having an angle of intersection 120°, by a circular curve of 600m radius. Calculate -

(a) Tangent distance, (b) Length of long chord, (c) Apex distance, (d) Middle ordinate, and (e) Degree of curve. **[Ans. 346.44; 600m; 92. 82m; 80. 40m and 2°51'53"]**

Problem 2. Stations A,B and C in the working of a seam are to be joined by a circular curve. The co-ordinates in chains B and C in relation to A are :

Station	Latitude	Departure
B	N 5.76	E 2.15
C	S 7.50	E 8.00

Calculate the radius of the circular curve to pass through the stations A, B and C.

[Ans. 7.854 Chains]

Problem 3. The co-ordinates in metres of two points B and C with respect to a point A are : B 536.23N, 449.95E, and C 692.34N, 1336.28E, Calculate the radius of a Circular curve passing through the three points and verify the result by construction. **[Ans. 1170 metres]**

Problem 4. The underground roadways AB and CD are respectively 21°50' and 300°10' and the co-ordinates of B and D are as follows :

B	5049.37m North	1762.67m West
D	5102.41m North	1263.97m West

Calculate (a) how far each roadway must be driven to meet the tangent points, and (b) the length of the new roadway BD. **[Ans. (a) 68.2m & 216.5m; (b) 628.1m]**

Problem 5. A circle passes through three points having the following co-ordinates in metres :

A	182 S	128 W
B	0	0
C	14 S	220 E

Calculate the diameter of the circle. **[Ans. 453.105 m]**

Problem 6. The centre lines of two straight portions of a railway have the following bearings :

Line	Azimuth
AB	150°
CD	60°

These straight lines are to be connected by a circular curve of 150m radius. Describe a method of setting out the curve and calculate (a) the tangent length and, (b) the length of the curve. **[Ans. 150m; 235.714m]**

Problem 7. A, B and C are three points which have to be connected by a curve of regular radius. The co-ordinates in metres of these three points are :

A	929.9 N	133.3 W
B	1165.6 N	325.4 E
C	884.5 N	633.7 E

Calculate the radius of the curve. **[Ans. 410.09m]**

Problem 8. Following are the co-ordinates of two points B and C in relation to a point A :

B. 117m South and 276m East from A.

C. 477m South and 452m East from A.

Plot the positions of three points to a suitable scale and find by construction the radius of a circular curve passing through the three points. Thereafter from the measurements taken from your drawing calculate the length of the curve from A to C.

[Ans. R = 506m; θ= 81°15'; 717.84m]

Problem 9. Calculate the super-elevation required for a curved railway track having 90m radius, distance between centres of the rail 1.225 metres and the speed of train 20.5 km per hour.

[Ans. 4.5 cms.]

Problem 10. A circle passes through three points having the following co-ordinates in metres :

Points	Latitude	Departure
A	0	0
B	182 N	128 E
C	168 N	348 E

Calculate the radius of the circle and verify your result by plotting to a suitable scale.

[Ans. 226.56m]

Problem 11. Two roadways having bearing of 20° and 80° respectively are to be connected by a circular curve of 50 metres radius. Calculate the distance from the point of intersection of the two roadways to the centre of curvature and the tangent distance. **[Ans. 50m; 86.6m]**

Problem 12. The underground roadways AB and CD are to be connected by a level roadway having a radius of 60 metres. The Azimuth of AB and CD are 30° and 310° respectively and the co-ordinates of B and D are : B 1515m North, 530m West; and D 1530m North, 380m West. Calculate how far each roadway must be driven to meet the tangent points.

[Ans. 38.08m; 52.8m]

CHAPTER III

TRIANGULATION, TRILATERATION AND TRAVERSING

The object of surveying is to accurately determine the relative positions of a system of widely separated points on the surface of the earth, and also their absolute positions. The relative positions are determined in terms of the azimuths and lengths of the lines joining them, and absolute positions in terms of latitudes and longitudes and elevations above mean sea level. The geodetic points so determined after measurement, computation and adjustment constitute a geodetic network to furnish the most accurate control for a more detailed survey of the intervening country. The horizontal positions of points in a network developed to provide accurate control for subsidiary surveys can be obtained by-

1. Triangulation,
2. Trilateration, and
3. Traversing.

1. **Triangulation :** Areas of considerable extent can not be surveyed precisely and satisfactorily by traversing methods in which the accuracy is limited by difficulties of precise linear measurement. Large areas are therefore usually covered by network of interlacing triangles. If all the angles of the triangles are very carefully observed and at least one of the sides called the base line is measured with considerable care and accuracy, then the lengths of all the remaining sides can be obtained from trigonometric formula. Furthermore, if the direction of one side is known, the direction of all other sides of all triangles can be determined and thus the position of each staton with reference to a predetermined point in the area surveyed can be computed by co-ordinates. This method of surveying is called *triangulation.*

The method of triangulation requires a considerable amount of precise angle measurement with a minimum amount of distance measurement. The base line must be measured with extreme precision, since errors propagated through the system originate with these measured lines. The precise linear measurements are the most costly and labour-consuming surveying operation. The advent of tellurometers and geodimeters has made it possible to obtain distances with highest precision directly within a very short range of time.

2. **Trilateration :** Since the advent of the long-range EDM's, a new method of surveying, called *trilateration*, has been adopted to combine with triangulation in order to extent horizontal control. Trilateration is based on the trigonometric proposition that if the three sides (laterals) of a triangle are known, the three angles can be computed. This accomplishes the same thing as

triangulation, namely, the determination of all the sides and all the angles in the triangle. Directions of the lines and positions of the points in the triangle can then be computed as in traverse and triangulation.

3. **Traversing :** *Traversing* is that type of survey in which the framework consists of a series of connected lines, the lengths and directions of which are measured with a chain or tape and with an angular instrument respectively, the stations of a traverse are so arranged that the lines joining them are continuous and their relative positions are fixed by measuring the length and directions of the lines, legs or drafts. The accuracy of a particular traverse is dependent upon the degree of refinement of instruments and methods of observation and these, in turn, governed by the purpose of the traverse.

Traverses are classified according to their form as (a) Closed traverse, when lines form a circuit which ends at the starting point, and (b) Open traverse, when the circuit does not close either polygonally or between points previously fixed by a more accurate survey. In closed traverse, the work is, to some extent self-checking, its angular error being judged by comparing the initial and final bearing of the closing line and its closing error by comparing the initial and closing co-ordinates of the survey. The amount of these errors will generally indicate whether actual mistakes have been made in the observations. In open traverses there is no check on the co-ordinates or angular closing error and the risk of unknown errors and mistakes is greatly increased.

TRIANGULATION

Given the three angles and the length of one side of a triangle, the lengths of the other two sides can be calculated by the application of sine rule. This provides a basis for fixing accurately the relative positions of three points on the earth's surface both mathematically and on plans or maps. Moreover, if a number of survey stations are so arranged that the lines joining them form a series of interlocking triangles, all the angles which can be observed, it is necessary to know only the length of one side of a triangle to be able to calculate all the others in the system. This method of surveying is known as "triangulation" and the accuracy of the method is based on :

(a) The length of the measured side, known as the base line, and

(b) The measurements of the angles.

Angles can be measured more accurately than lines and, as the method only requires the determination of the length of one line of the survey (a side of one of the triangles), the line can be fixed under the most favourable topographical conditions. Every refinement can be used in the measurement of it, to obtain the highest degree of accuracy.

Triangulation is the most rapid method of fixing of precise system of control points and is invariably employed when the area to be surveyed is large. The sum of the three measured angles in each triangle should equal 180^0 and must be adjusted if they do not do so. Very large triangle on the earth's surface are spherical and the sum of the angles of a spherical triangle is 180^0 plus an amount known as "*spherical excess*". In triangulation required for mining surveying, adjustment for spherical excess and the special formulae used in the calculation of spherical triangulation can be ignored, as the error is small when relatively small areas of ground are considered.

CLASSIFICATION OF TRIANGULATION SYSTEM

The basis of the classification of triangulation figures is the accuracy with which the length and azimuth of a line of the triangulation are determined. Triangulation systems of different accuracies depend on the extent and the purpose of the survey. The accepted grades of triangulation are :

1. First order or Primary Triangulation.
2. Second order or Secondary Triangulation.
3. Third order or Tertiary Triangulation.

1. First-Order or Primary Triangulation : It is the most accurate and would, generally, form the basis for the surveying and mapping of a whole country. Every precaution is taken in making linear and angular measurements and in performing the reductions. The following are the general specifications of the primary triangulations :

1. Average triangle closure	:	Less than 1 second.
2. Maximum triangle closure	:	Not more 3 seconds.
3. Length of base line	:	5 to 15 Kilometers.
4. Length of sides of triangles	:	30 to 150 Kilometres.
5. Degree of accuracy in triangulation	:	1 in 500,000
6. Probable error of computed distance	:	1 in 60,000

2. Second-Order or Secondary Triangulation : The secondary triangulation consists of a number of points fixed within the framework of primary triangulation. The stations are fixed at close intervals so that the sizes of the triangles formed are smaller than the primary triangulation. The instruments and methods used are not of the same utmost refinement. The general specifications of the secondary triangulation are :

1. Average triangle closure	:	Less than 3 sec.
2. Maximum triangle closure	:	Upto 8 sec.
3. Length of base line	:	2 to 5 kilometres.
4. Length of sides of triangles	:	8 to 65 kilometres.
5. Degree of accuracy in triangulation	:	1 in 50,000
6. Probable error of computed distance	:	1 in 20,000

3. Third-Order or Tertiary Triangulation : It consists of a number of points fixed within the framework of secondary triangulation, and forms the immediate control for detailed engineering and other surveys. The sizes of the triangles are small and instrument with moderate precision may be used. The specifications for a third-order triangulation are :

1. Average triangle closure	:	6 seconds.
2. Maximum triangle closure	:	15 seconds.
3. Length of base line	:	0.5 to 3 km.
4. Length of sides of triangles	:	Upto 8 km.

5. Degree of accuracy in triangles : 1 in 5,000

6. Probable error of computed distance : 1 in 5,000

The Survey of India topo triangulation consist of a network of triangles formed by primary, secondary and tertiary triangulations, covering the whole of the country. All primary, secondary and tertiary ground stations have been permanently marked – in most cases by a concrete pillar on which a three-screw theodolite can be set up and is automatically centred, Beacon lamps, mounted on a stem, can be inserted into an iron pipe in the centre of the pillar.

Stations may also be fixed on buildings, colliery headgears, etc. and are marked by large mushroom headed brass bolts. Each such station is correlated, by measurements, to four smaller bolts or marks–known as witness bolts and witness marks, situated on parts of the structure which are unlikely to be disturbed by repair work. All the above type of stations are known as *downstations*. Some of the tertiary stations are '*upstations*', such as lighting conductors on tall chimney's, church spires, flagpoles, etc. which means that they have been observed but not occupied.

Underground workings have to be shown accurately on mine plans, to correct relationship with the overlying surface features. It is therefore necessary to establish certain surface points to a degree of accuracy only possible by triangulation. The positions of the surface reference stations and the centres of all mine shafts at the surface and reference points of underground surveys and also the boundaries of the mine and all surface features required to be shown shall be shown upon the Key and Master Plans in their correct positions relative to the Survey of India National Grid within the limits of error of survey and plotting required by the Code.

The position of every station points of triangulation with reference to the points of origin of the survey, calculated from an initial base line (or the Survey of India topo triangulation stations) shall agree with the position of that station point, calculated from a verification base line, within a limit of error not exceeding 1/5000th of the linear horizontal distance of the station point from the point of origin.

Every *traverse* made between station points of a triangulation and or for the determination of the boundaries of the mine or for determination of other important surface details, shall be closed polygonally and shall be made within limits of error as follows :

(i) The total angular error of the traverse shall not exceed $30\sqrt{n}$ second when 'n' is the number of observing stations (including the initial and closing stations).

(ii) The error of closure (calculated by co-ordinates) after distribution of the total angular error shall not exceed 1 / 3000th of the sum of the horizontal lengths of the drafts of the traverse.

TRIANGULATION FIGURES OR SYSTEMS

A triangulation gigure is a group or system of triangles such that any figure has one side and only one, common to each of the preceding and following figures. The common figures are

1. Single chain of triangles.
2. Double chain of triangles.

3. Central point figures.
4. Quadrilaterals.

The following factors should be considered while selecting a particular figure :

(a) The figure should be such that the computations canbe done through two independent routes.

(b) At least one, and preferably both routes of the figure should be well conditioned.

(c) All the lines in a figure should be of comparable length.

(d) The figures should be such that least work may secure maximum progress.

(e) Complex figures should not involve more than about twelve conditions.

WELL-CONDITIONED TRIANGLE

There are various triangulation figures and the accuracy attained in each figure depends upon (i) the magnitude of the angles in each individual triangle, and (ii) the arrangement of the triangles. Regarding the shape of the triangle should be such that any error in the measurement of angle shall have a minimum effect upon the length of the calculated side. Such a triangle is then called a *well-conditioned triangle*.

In a triangle, one side is known from the computations of the adjacent triangle. The error in the other two sides will affect the rest of the triangulation figure. In order that these two sides be equally accurate, they should be equal in length. This can be attained by making the triangle isosceles.

To find the magnitude of the angle of a triangle, let A, B and C be three angles and a, b and c be the three opposite sides of an isosceles triangle ABC. Let AB be the known side, and BC and CA be the sides of equal length to be computed. Evidently $\angle A = \angle B$.

By sine formula, $a = c \frac{\text{Sin A}}{\text{Sin C}}$ ..... (i)

Let δA = the error in the measurement of $\angle A$

and δa_1 = corresponding error in the side a.

Differentiating (i) partially, we get

$$\delta a_1 = \frac{c \text{ Cos A } \delta A}{\text{Sin C}}$$

$$\therefore \frac{\delta a_1}{a} = \frac{\text{Cos A}}{\text{Sin A}} \delta A = \delta A.\text{Cot A} \quad \text{..... (ii)}$$

Similarly. let δC = the error in the measurement of $\angle C$

δa_2 = corresponding error in the side a.

Differentiatiang (i) partially, we get

$$\delta a_2 = -C \frac{\text{Sin A Cos C. } \delta C}{\text{Sin}^2 C}$$

$$\therefore \frac{\delta a_2}{a} = \frac{\text{Cos C}}{\text{Sin C}} \delta C = -\delta C \cot C \quad \text{..... (iii)}$$

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If δA and δC = probable errors in angles = $\pm \beta$

the probable fraction error, $\frac{\delta a}{a'}$ in the side a

$$= \pm \beta \sqrt{\cot^2 A + \cot^2 C}$$

This is minimum when $\cot^2 A + \cot^2 C$ is minimum.

But $C = 180^0 - A - B = 180^0 - 2A$

$\therefore \cot^2 A + \cot^2 2A$ with respect to A and equating it to zero, we get after reduction,

$$4 \cos^2 A + 2 \cos^2 A - 1 = 0$$

From which $A = 56^0 14'$ approx.

Hence **the best shape of triangle is isosceles with base angles equal to** $56^0 14'$. However, from practical consideration, an equilateral triangle is most suitable. In general, however, triangles having an angle smaller than 30^0 or greater than 120^0 should be avoided.

ROUTINE OF TRIANGULATION SURVEY

The routine of triangulation survey generally consists of the following operations :

1. Reconnaissance
2. Erection of signals and towers
3. Measurement of base lines
4. Measurement of horizontal angles
5. Astronomical observations, and
6. Computations.

Since the basic principle of surveying is working from whole to part, reconnaissance is very important in all types of surveys. The following operations are required for an efficient reconnaissance :

1. Examination of the country to be surveyed.
2. Selection of suitable sites for base lines.
3. Selection of suitable positions for triangulation stations.
4. Determination of intervisibility and height of stations.
5. Collection of miscellaneous information regarding communication of water, food, labour and guides, etc.

Wherever possible, help should be taken from the existing maps. The reconnaissance survey requires great skill, experience and judgement on the part of the party chief. **The following instruments are generally used for the survey :**

1. A theodolite and sextent for measuring of angles.
2. Prismatic compass for measurement of bearings.
3. Aneroid barometer for ascertaining elevations.
4. Steel tape.

5. Good telescope or powerful field glass.
6. Heliotropes for testing intervisibility.
7. Drawing instruments and materials.
8. Guyed ladders, ropes, creepers, etc. for climbing trees.

SELECTION OF TRIANGULATION STATIONS

The selection of stations is based upon the following considerations :

1. The stations should be intervisible.
2. They should form well-shaped triangles. As far as possible, the triangles should be either isosceles with base angles of about + 56^0 or equilateral. No angle should be smaller than 30^0 or greater than 120^0.
3. The station should be easily accessible, supplies of food and water are easily available and camping ground or nearest suitable accomodation is available.
4. The length of sight should neither be too small nor too large. Small length of sight results in errors due to centering and bisection, while large line of sight makes the signal too indistinct for accurate bisection.
5. They should be in commanding situations so as to serve as to control of the subsidiary triangulation and for possible future extension of the principal system. The stations of the subsidiary triangulation should be such that they are useful for detail surveys.
6. In heavily wooden country, the stations should be so located that the cost of clearing and cutting and of building towers are minimum.
7. The stations should be situated so that lines of sight do not pass over towns, factories, furnace etc., nor graze any obstruction, so that the effects of irregular atmospheric refraction is avoided.

BASE LINE MEASUREMENT

The measurement of base line forms the most important part of the triangulation operations. The base line is laid down with great accuracy of measurement and alignment, as it forms the basis for computations of triangulations system. The length of the base line depends upon the grades of the triangulation system. Apart from the main base line, several other check bases are also measured at some suitable intervals. In India, ten bases were used, the length of nine bases vary from 6.4 to 7.8 miles and that of the tenth base is 1.7 miles.

SELECTION OF SITE FOR BASE LINE

The following points should be taken into consideration while selecting the site :

1. The site should be fairly level, or uniformly sloping or gently undulating.
2. It should be free form obstruction throughout the whole of the length. The line clearing should be cheap in both labour and compensation.
3. The ground should be firm and smooth.
4. The extremities of the base should be intervisible at ground level.

5. The site should be so selected that well-shaped triangles can be obtained in connecting the end stations of the base to the main triangulation stations.

METHODS FOR MEASURING THE BASE LINE

The base-line may be measured in two ways :

1. The measurements are made off the ground by a steel or invar **tape hanging in catenary** between two supports with additional intermediate supports. This method is specially advantageous in rough and hilly country where a base of reasonable length on level ground may be difficult to find. It is, however, slower and needs special apparatus. Calm weather conditions are also essential.

2. In level or slightly inclined countries, the line to be measured between two pegs is aligned with a theodolite and the tape is placed over it for measurement. The measurement is made along the ground.

1. Measurement of Base line by Tape in Catenary

If a wire, which is of uniform weight along its length is suspended freely between simple supports then it hangs in catenary. If the length of the catenary is known then the distance between the supports can be calculated. This fact is used, in this method of base line measurement. Normally a tape 100 metres long is used. This is suspended above the ground between two measuring tripods, the tape used is made of invar steel because of its very low coefficient of expansion. Invar is an alloy of nickel and iron and it was found that when there is about 37 per cent of nickel in the alloy then the coefficient of expansion is a minimum. The coefficient of expansion is not solely dependent on the nickel content since varying amount of manganese and carbon will also affect it. It is possible to produce an invar steel with a negative coefficient of expansion. The coefficient of invar steel is about 12×10^{-6} par degree C.

The equipment used consists of several measuring tribrachs which can be attached to tripods, two straining tripods so that the tape can be correctly tensioned. The measuring head has a fiducial mark which is a knife-edge, supported on a tribrach and shifting head. Above the fiducial mark can be fitted either a magnifying eyepiece for reading the tape, a target, or a theodolite for lining in the tridods and measuring the vertical angle.

The first measuring tripod is set over one end of the base line and accurately centred over the reference mark using the optical plummet. The theodolite is set on this measuring head and the far end of the base line is sighted. If it is a long base line then control points to assist alignment must be made about every half Km. The telescope is thus brought into line and second measuring tripod is set up exactly in line, using the target and getting fine adjustment with the shifting head.

If three or more measuring tripods are available then these can be set in line at the ends of their appropriate bay. The two straining tripods are set up externally to the bay to be measured. The pulleys of the straining tripods have screws in allow transverse movement to ensure correct alignment of the tape. The tape is placed in position, ensuring there is no twisting, the straining weights are applied, giving the standard tension for the tape. The ends of the tape are read simultaneously through the magnifying eyepiece of the measuring head. The temperature is

recorded. For very accurate work the thermometer used should be enclosed by a cover of the same material, colour, and brightness as the tape being used. The tape is than moved a short distance one way and two further readings made. It is then moved a short distance in the opposite direction and further readings taken. This is repeated until sufficient readings have been taken. The tape is then transferred to the carries. The straining trestles are moved into position for measuring the second bay.

The procedure is repeated. If three measuring tripods are in use the theodolite is placed on the third one to line in the fourth position using the tripod which had been in the first position. This method is followed throughout the length of the base line. The line should be measured at least once in each direction.

2. Measurement by Tape along the Ground

In this method which is the least accurate giving results accurate to about 1 in 10000, the tape is supported throughout its length by the ground. The end of the base line must be accurately fixed and would normally be carried on concrete pillars in an area which is as level as possible and which will give the longest base line. The horizontal length between these two marks must be accurately determined. The tape in use may either be 20 m or 100 m long. A certain amount of preparatory work is required. The line will have to be measured in bays which are equal or slightly less than the length of the tape. At each end of these bays, must be fixed wooden pegs driven into the ground level with a soft metal plate (zinc) fixed to the top. These wooden pege must be aligned by a theodolite and fixed in their correct longitudinal position by use of the tape.

When the plates are in position a longitudinal line is scratched on each plate using the theodolite to ensure that this scratch is truly in line. The tape is then applied to the first bay length. The man at rear holds the tape such that the end of the tape is on the plate. He does not need to ensure coincidence with the station mark, since error may arise if this is attempted.

There will thus be a correction, which may be either positive or negative, to be applied to the measured bay length.

The man on the forward end of the tape will have spring balance which has been attached to the tape. He can, therefore, tension tape to either the standard tension or to given tension. When the tension is applied the surveyor gives the directions to scratch the plate at either end opposite the mark on the tape. The temperature is determined by thermometer and recorded. The tape is then carried forward carefully and the same procedure adopted at the next bay. There will be a correction at each bay length. This is measured by a small scale and its appropriate sign is noted. A levelling must then be carried out to determine the reduce level of each peg. This is required to correct the measured length for slope. If the bay lengths are on a fairly steep slope then the inclined angles may be measured by clinometer. If there is a decided change in gradient between different parts of the bay then the gradient of each must be noted. This procedure is followed throughout the line and the final bay length or part of a bay is also measured. This completes the field work involved other than repeating the measurement several times to obtain a mean measurement.

The final value for the whole base line is obtained by applying certain corrections to the mean measurement of each bay, and when the length of the base (from the sum of these corrected

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bay lengths) has been found the remaining corrections are made to this value :

(a) *Those to be applied to each bay for -*

(i) Standard or Absolute length,

(ii) Temperature,

(iii) Tension (if the standard tension not applied),

(iv) Sag (if the tape is used in catenary),

(v) Slope,

(vi) Horizontal alignment.

(b) *Those to be applied to the sum of the corrected bay lengths for -*

(i) Mean height above Mean Sea Level, and

(ii) Local Scale Factor.

CORRECTION REQUIRED IN THE BASE LINE MEASUREMENT

The following corrections are to be applied in the field measurements of baseline in order to get its true length :

1. **Correction for Standard or Absolute length :** The field tape is liable to suffer some elongation due to continual elongation to which it is subjected in the field and so far accurate work field tape should be tested with a standard tape before and immediately after each day's work in the field. The length of the field tape in terms of the standard tape may then be deduced. The measured length is too great when the tape is short or too less when the tape is long. The *correction is therefore either negative or positive.* Thus -

$$\text{Correction} = \frac{LC}{l}$$

When L = Measured length of the line.

C = Correction to the measuring tape.

l = Nominal length of measuring tape.

2. **Correction for the temperature :** Steel tape expands as the temperature rises and contracts as the temperature falls. *The correction is positive or negative* according as temperature during measurement (T) is greater or less than temperature of standardisation (T_0).

Correction = $L \alpha (T - T_0)$

Where α = Co-efficient of thermal expansion.

For steel α = 0.00012 per 1°C, and for invar 0.0000010 per 1°C.

3. **Correction for Tension :** A tape stretches when pull is applied. If the pull applied to the tape is different than the standard then the correction must be applied.

$$\text{Correction} = \frac{(P - P_0)L}{AE}$$

When P = Pull applied during measurement in Kg.

P_0 = Standard pull in Kg.

L = Measured length in metres.

A = Cross-sectional area of tape in Sq. cm.

E = Modulus of elasticity of the tape.

For steel E = 21 x 10^5 Kg. per sq. cm.; for invar 154 x 10^5 kg. per sq cm.

4. **Correction for Sag :** A tape sags between the points of supports. The correction for sag is the difference in length between the arc and its chord. The effect of sag on the tape is to make the measured length too long and so this *correction is always negative.*

$$\text{Correction} = \frac{L (WL)^2}{24P^2}$$

Where L = Distance between support in metres.

W = Weight of tape in Kg per metre run.

P = Applied pull in Kg.

5. **Correction for Slope :** When the supports are not at some level, corrected slopping distance is reduced to the horizontal by subtracting the correction required.

$$\text{Correction} = L (1 - \cos \theta) = \frac{h^2}{2L}$$

Where L = Length of slope in metres.

θ = Grade of the section.

h = Difference in elevation.

6. **Correction for Horizontal Alignment :** Error crops up if one end of the tape is out of line between the points measured. The distance between the points measured should be less and so the *correction is always negative.*

$$\text{Correction} = \frac{d^2}{2L}$$

When L = measured length in metres, and d = Amount of deviation of the point out of line.

7. **Reduction to Mean Sea Level :** For convenience of comparison with National Grid Co-ordinates, the length of the baseline is reduced to its equivalent length at mean sea-level datum. The Correction is negative.

$$\text{Correction} = \frac{Lh}{R}$$

Where L = Measured length of base in metres.

h = Average height of base above M.S.L. in metres.

R = Mean radius of earth in metres.

8. **Correction for Local Scale Factor :** A short traverse on a plane surface can be easily represented on a plan with grid lines without any apparent distortion, but in the case of larger areas the curvature of the spherical surface of earth greatly affect the actual shape of the map. Objects on the curved surface of earth can not be represented on a flat sheet of paper in the same relative positions which they occupied on the globe. A slight difference between the grid distances occurs because the meridians converge and the grid lines are drawn parallel to one

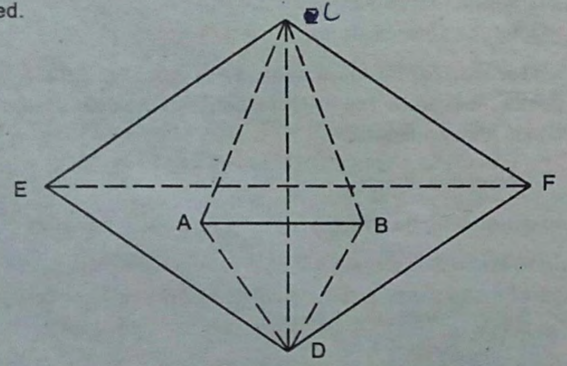
central meridian. Therefore, in precise surveys of larger areas the distances computed from co-ordinate will require a correction by a factor known as **"local scale factor"**. The value of the local scale factor will vary with geographical position and depending upon the departure east or west from the central meridian.

PROLONGATION OF BASE LINE

The measurement of a long baseline may sometimes be difficult due to obstacles or the level ground being of limited extent. In such case, only a portion of the base line accurately measured which is then prologned by ranging lines in continuation of it, at one or both ends, until a suitable length is obtained to give well-conditioned angles from each of its ends to the points to be fixed. The lengths of the additional lines are calculated from the measured base and the observed angles.

To prolong the measured base line AB, two stations C and D, visible from A and B, and from each other, are chosen on opposite sides of AB, to form well-conditioned triangles as nearly equilateral as possible. The angles at each of the four points, subtending by the remaining three, are then observed. The length of CD is calculated from the different triangles and a mean result is adopted.

In this way, by the use of such stations as E and F, the line CD may be extended and the distance EF computed.



Double Extension of Base Line

Proceeding in this manner, a suitable extended base may be determined. It should be noted that the extension of base line must be treated as a single problem and not as a series of separate problem. Thus station C and D should not be finally fixed until the positions of E and F and the further points are definitely located, because the final positions of the stations forming the extended base should be the stations of the triangulation network.

DOUBLE EXTENSION OF BASELINE

The measurement of a longer baseline may be difficult due to obstacles. Furthermore, the accuracy of linear measurements maybe adversely affected by poor topographical conditions of

the area. It is therefore better to measure a short base line which is extended by well-conditioned triangles.

In a triangulation network consisting of triangles CDE and CDF, the longer line EF is not directly measured to avoid obstacles or poor topographical conditions of the area. A short baseline AB is very carefully measured and two intervisible stations C and D, both visible from A and B and which can form well-conditioned triangles with AB, are selected. All the necessary angular measurements are taken so as to calculate the first extension CD. In a similar way, the line CD is further extended to EF. Hence, EF is the double extension of the measured base line AB.

BASE LINE MEASUREMENT BY USE OF EDM

The use of electronic distance measuring instruments (EDM's) eliminate the need for elaborate baseline preparation. The general requirement for a base line that is to be measured by means of one of these instruments is a clear line of sight between the two ends of the base. A base line may be chosen between two prominent triangulation stations since the measurements with an EDM is completely independent of the character of the intervening terrain.

The time required to make the necessary instrument readings during the actual measuring operations is independent of the length of the base line. When an EDM is used under normal conditions, the instruments can be set up and aligned and all the readings can be made in about an hour. There is a tremendous saving in time, especially when base lines 30 to 45 Km in length are considered.

Check bases tend to strengthen the entire triangulation network. Because of the ease with which bases can be measured by using the EDM, check bases can be selected at more frequent intervals than when base lines are measured with tapes.

The accuracy of the EDM is limited by a constant uncertainty such as 5 mm to 10 mm at close range. As the measured distance is increased, this constant value becomes relatively inconsequential. Beyond 500 to 1000 m, all of the currently available EDM's will give relative accuracies of better than 1 in 25000. This is very difficult to obtain by taping. If meteorological conditions at the time of measurement are known with sufficient accuracy, then all but the very short-range instruments are capable of relative accuracies of 1 in 100,000 or greater.

The corrections for meteorological conditions i.e. the effects of atmospheric conditions such as temperature and pressure, and instrumental corrections should be applied to the measured length to obtain the corrected slope distance. In order to reduce the slope distance to sea level as required, either the reciprocal vertical angles are measured at the two ends of the line or the elevations of the two ends of the line must be known.

ADJUSTMENT OF HORIZONTAL ANGLES

After measuring the horizontal angles in a triangulation survey the following two-step adjustments are made in order to determine their most probable values -

1. Station Adjustment.

2. Figure Adjustment.

1. Station Adjustment :

If angles are measured by closing the horizon at a station, the angles around each of the station are adjusted so as to fulfil the condition that their sum must be equal to 360°. The discrepancy should be proportionately distributed among the angles at the station.

2. Figure Adjustment :

Figure adjustments are made using the adjusted values obtained by station adjustment. The figure under consideration may be -

(a) **The Triangle :** This is the simplest figure and the only geometrical condition to be satisfied is the sum of the three angles of a triangle equal 180°. The discrepancy should be equally distributed to each of three angles in a triangle. If different weights are assigned to the angles, the discrepancy must be distributed among the angles in proportion to the reciprocal of their weights.

(b) **The Quadrilateral :** In a quadrilateral with one observed diagonal two triangles are involved and conditions to be satisfied are -

(i) The sum of the angles of each triangle = 180°

(ii) Angles ABD + DBC = Angle ABC, and

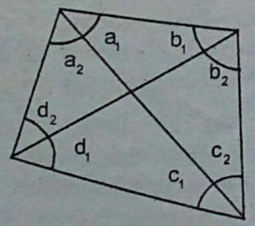
(iii) Angles ADB + BDC = Angle ADC.

Any discrepancy should be equally distributed to the Angles under consideration.

(c) **The Crossed or Braced Quadrilateral :** This is the geometrically strongest figure of all where all the angles in the figure are measured. It is comprised of four triangulation stations in which there are eight measured angles. The term *braced* comes from the fact that the two diagonal lines are observed, thus bracing the four-sided outer figure.

A. Angular Conditions.

1. Angles $a_1 + b_1 + b_2 + c_2 + c_1 + d_1 + d_2 + a_2 = 360^0$
2. Angles $a_1 + b_1$ = Angles $c_1 + d_1$
   Angles $a_2 + d_2$ = Angles $b_2 + c_2$
3. Angles $a_1 + b_1 + b_2 + c_2 = b_2 + c_2 + c_1 + d_1$
   $= c_1 + d_1 + d_2 + a_2$
   $= d_2 + a_2 + a_1 + b_1 = 180^0$



Adjustments :

1. Using the values obtained by usual station adjustment the sum of eight angles are found out which should be 360°. If the sum is more or less then 360°, each angle should be adjusted by one eighth of the discrepancy.

2. From the values of the angle so obtained, the difference between the sums $(a_1 + b_1)$ and $(c_1 + d_1)$ is calculated and the discrepancy is equally distributed among the four angles under consideration. If $(a_1 + b_1)$ is greater than $(c_1 + d_1)$, the sign of the correction to a_1 and b_1 is minus, and that for c_1 and d_1 is plus and vice versa. The discrepancy for $(a_2 + d_2)$ and $(b_2 + c_2)$ should also be treated in a similar way.

3. The three angles of a triangle should sum up to 180°. This condition will automatically satisfy if the first two adjustments are made.

B. Side Conditions

The length of any one side of a crossed-quadrilateral, as calculated from the known lengths of any of the other sides should always be the same. The adjusted values of the angles are tested to satisfy the following side equation -

The sum of the log sines of the left hand angles must be equal to the sum of the log sines of the right-hand angles.

Here the correction per angle is obtained by dividing the discrepancy by sum of differences for 1 second of all the angles. The corrected value of angles are found out after making necessary correction in value of log sines.

(d) **The Polygon :** In a fully observed polygon of 'n' sides the conditions to be satisfied are-

(1) The sum of central angles must be 360°.

(2) The sum of outer angles at all stations must be (2n + 4) right angles.

(3) The sum of angles in each triangles must be 180°.

(4) The sum of log sines of right-hand and left-hand angles must be equal.

To determine the most probable values of the corrections, the method of correlats may be used. The adjusted values of angles may then be found by applying the corrections to the observed values.

COLLIERY TRIANGULATION

The procedure for carrying out a systematic triangulation survey in a colliery may be divided under the following heads -

1. To study any available surface plan of the area to be surveyed to select most probable positions of triangulation stations and site for a base line.

2. To reconnoitre the area and prepare a rough sketch showing important surface features within the area.

3. Select suitable positions for triangulation stations and most favourable site for the base line.

4. To determine the amount of cutting and cleaning necessary to make the line of sights free from obstruction.

5. To measure the base line.

6. To determine the true meridian and absolute positions of stations.

7. To measure horizontal angles.

8. To measure one check base for verification.

9. To compute for - (a) adjustments of angles, (b) length of sides of triangles, and (c) latitude and longitude of stations.

10. To plot the triangulation frame work.

11. To carry out a traverse survey to locate all important surface features and plot the same within the triangulation framework.

Procedure : Underground workings have to be shown accurately on a plan, in correct relation with overlying surface features. It is, therefore, necessary to establish certain surface points to a degree of accuracy only possible by triangulation. The only common points between surface and underground are the mine entries and this must be fixed precisely.

All workings should be correlated with the Ordnance Survey triangulation so that there is a common relationship between them. If sufficient O.S.stations are not available in the area it will be necessary to carry out a minor triangulation to include two or more O.S. stations and the main points on the surface on the area, which require to be located.

An examinatiion of an Ordnancae Survey map or any surface plan of the area, coupled with local knowledge, should enable probable positions of triangulation stations and site for the base line to be fixed from which suitable observations can be made. The need for well-conditioned triangles should be borne in mind when preparing the preliminary triangulation layout.

The whole area is reconnoitered and intervisibility of stations checked. In mining areas, triangulation stations may be affected by mining subsidence so any suspect stations should be disregarded. The stations selected in a manner to cover the whole property including dwellings, streams, shafts, inclines, roads and other important surface features. Triangles should be well-conditioned and sides should vary from 300m to 500m. The stations thus selected for triangulation survey over the area are then laid out.

The base line should be on firm ground, fairly level or uniformly sloping and free from obstructions throughout its length. This should be centrally situated over the area free from effect of subsidence.

The best site for the base line may be straight stretch of nearly level road or a railway line, which can be accurately measured.

The ends of the base line are accurately fixed and the base line is measured with greatest care. The direction of the base line in ralation to the true meridian is determined and all the horizontal angles at each station are then measured and recorded. One check base at other end of the survey is very carefully measured so as to find out the accuracy of whole work after computation.

Co-ordinates of triangulation stations are calculated and after necessary adjustments they are plotted on a plan. The co-ordinates of triangulation stations form a series of control points whose positions are fixed with very great accuracy.

A theodolite traverse survey is carried out from triangulation stations to locate all the

required surface features to be shown on the plan within the triangulation area. The traverse should be closed on nearby triangulation station and the details picked up are plotted to the same scale on the same plan of the triangulations survey.

Accuracy

In colliery triangulation an accuracy of 1 in 5000 for calculated distance between two stations should be attained. For this accuracy, the triangle closure should not exceed ± 15 seconds which is the maximum permissible error.

TRILATERATION

Principle : The principle of trilateration is based on the trigonomertical proposition that if the three sides (laterals) of a triangle are known then all the angles of the triangle can be computed from the following trigonometric formulae :

$$\text{Let } s = \frac{a+b+c}{2}$$

$$\text{Sin}\frac{1}{2}A = \sqrt{\frac{(s-b)(s-c)}{bc}}$$

$$\text{Cos}\frac{1}{2}A = \sqrt{\frac{s(s-a)}{bc}}$$

$$\text{Tan}\frac{1}{2}A = \sqrt{\frac{(s-b)(s-c)}{s(s-a)}}$$

$$\text{Sin } A = \frac{2\sqrt{(s-a)(s-b)(s-c)}}{bc}$$

$$\text{Cos } A = \frac{b^2+c^2-a^2}{2bc}$$

The other angles B and C can also be determined in the same way.

Procedure : The trilateration system consists in construction on the ground of a continuous chain of triangles and braced quadrilaterals in which all the sides and diagonals are measured directly, using the electronic distance measuring instruments (EDM's), without measuring any horizontal angles in the network. The angles are obtained from trigonometric formulae and then the co-ordinates of all the control points are computed. In order to maintain the accuracy of the azimuths of the lines in trilateration net, astronomical observations are made at selected stations. These measured azimuths impose conditions that must be satisfied in the adjustment process.

Trilateration is frequently combined with triangulation in order to strengthen a net that may have serious deficiencies in geometric conditioning, or trilaterated lines are used as check bases in a chain of figures. It is possible to observe a net completely as both triangulation and trilateration. Such a hybrid network imposes several rigid geometric conditions that must be met in the adjustment of the net. However, in certain instances this rigidity is necessary. The measurement of very small displacement due to fault movement should be made over a fairly large areas periodically with the same degree of accuracy and reliability in order to reflect the earth movement. The internal accuracy and reliability in such network is greatly enhanced by a triangulation-trilateration network.

Corrections : Before the lengths of the lines in trilateration net can be used in any subsequent computations, their slope lengths determined by the instrument and corrected for atmospheric conditions must be reduced to the corresponding sea-level distances. To determine the slope corrections, auxiliary data either reciprocal vertical angles at the two ends of each line or the elevations of the two ends of each line must be measured. The conditions of the atmosphere that have an effect on the velocity of propagation of light and microwaves are the air temperature, the atmospheric pressure, and relative humidity. The temperature and relative humidity, in turn, define the vapour pressure in the atmosphere. Such effects of atmospheric conditions are handled in various ways by different EDM systems. The corrections are of little consequence for short distances using the light-generating EDM's. For longer lines an error of 10^0 C in the effective temperature of the beam path will introduce a relative error of 10 ppm; and an error of 25 mm of mercury in measuring atmospheric pressure will also introduce a relative error of 10 ppm. Corrections are either computed based on meteorological data determined at the time of measurement, or else the instrument circuitry is modified to take the atmospheric conditions into account. In the instrument H-P 3800 distance meter of M/s. Hewlett Packard Co. environmental corrections are dialed into the power unit. This changes the modulating frequency in order to maintain a constant wavelength at any temperature and pressure. In case of microwave EDM's, the partial pressure of water vapour obtained by the wet and dry bulb thermometer readings must be determined accurately. A 2 mm error in vapour pressure or a 1.5^0C error in the difference between dry and wet bulb temperatures will produce a relative error of about 10 ppm at normal temperatures. This relative error increases with an increase in air temperature. Assuming that the meteorological conditions have been measured satisfactorily, corrections to measured distances are easily made with the aid of various charts, or monographs that are furnished with the instruments.

Adjustment of Trilateration : Although there are several ways in which a quadrilateral or a network of figures in a trilateration system can be adjusted, one that can be used when extreme accuracy is not required is as follows :

The angles in each of the triangles are computed from the lengths of the sides, and then these angles are adjusted following the rules for adjustment of quadrilaterals. As a check on the computed angles, the three angles in any triangle must add up to 180^0 exactly, whether or not spherical excess exists. If a spherical angles in a large triangle are needed for computation of spherical co-ordinates, one-third of the spherical excess is added to each computed angle.

When high accuracy is needed, a least-squares method of adjustment must be followed. One such method, called variation of co-ordinates, involves the prior determination of a set of approximate co-ordinates of all the points to be adjusted.

One difficult problem in adjusting a trilateration net is that assigning weights to the measured lengths. The standard errors for relatively short lines may be just as great as those for long lines because of the inherent errors in the measuring system. The order of accuracy of a long line is higher than that of a short line because these inherent errors are rather constant. For example, suppose that an inherent error of 5 cm exists in the EDM. Then the accuracy ratio for a line 1 km long, is 1 part in 20,000, whereas that for a line 20 km long, is part 1 in 400,000. If the standard

error of each line can be determined by repeated measurement of each line, then the weights can be assumed to be inversely proportional to the squares of the standard error. For a limited net with relatively few lines, all of which have lengths of the same order of magnitude, a unit weight can be given to each line.

Standards of Accuracy

(1) *Lengths measurements* :

Network stations less than 10 km,		
Other surveys less than 0.5 km.....	1 in	1,000,000
Principal stations less than 0.5 km.....	1 in	250,000
Principal stations less than 0.25 km	1 in	150,000

(2) *Closure in position* :

Network stations less than 10 km,		
Other surveys less than 3 km.....	1 in	100,000
Principal stations less than 0.5 km.....	1 in	10,000
Principal stations less than 0.25 km.....	1 in	5,000

TRAVERSING

Triangulation and trilateration networks are the most accurate methods for fixing the horizontal control points. But in densely wooded country or very flat country such methods are physically impossible or very expensive for erecting tall becons or signals in order that the control points be visible. In such conditions it is advantageous to construct control nets using the method of traversing. The results of field measurement related to traverse will be a series of connected lines of known lengths related to one another by known angles and bearings. The lengths of the lines are determined by direct measurement of horizontal distances, by slope measurements, or by indirect measurement such as based on the methods of tacheometry.

The **purpose of the traverse** may be as follows :

(1) *On the surface,* the traverses are used for the location of important features such as shafts, boundaries, reservoirs, buildings to be protected against subsidence, railways, canals, rivers; for the setting out buildings, sidings, railroads, highways and other engineering works; for determining the area within the confines of a boundary; for establishing control for mapping; and for gathering data regarding earthwork quantities in construction work.

(2) Belowground, it is used for correlation work; for the fixing of trunkroadways; for aligning roads; and for survey work required for establishing crossmeasure drifts and other important underground roadways.

In genreal, traverses are of two classes. When the traverse originates either at a point of known horizontal position with respect to a horizontal datum or at an assumed horizontal position, and terminates at an unknown horizontal position, the survey is termed as open traverse. A

traverse may be termed as closed traverse, which can be described in any of the following three ways : (1) It originates at an assumed horizontal position and terminates at that same point; (2) it originates at a known horizontal position with respect to a horizontal datum and terminates at that same point; (3) it originates at a known horizontal position and terminates at another known horizontal position. A known horizontal position is defined by its location on or in relation to a fixed boundary.

An open traverse is usually run for exploratory purposes. There are no arithmetical checks on the field measurements. Since the figure formed by the surveyed lines does not close, the angles can not be summed up to a known mathemetical condition. None of the positions of the traverse stations can be verified, since no known or assumed position is included except that of the starting station. To strengthen an open traverse, several techniques may be employed. Each distance can be measured in both direction and can be checked by using the stadia hairs of theodolite or by using EDM. The measurements of the angles at the stations can be repeated. However, the open traverse does not reveal mistakes or errors and the results are always open to doubt.

A traverse that closes on itself immediately affords a check on the internal accuracy of the measured angles, provided that the angle at each station has been measured. A closed traverse gives an indication of the consistency of measuring distances as well as angles by affording a check on the positions closure of the traverse. Unless astronomical observations for azimuths have been made at selected station of a ttraverse that closes on itself, the only provision for verifying the directions of the lines is that afforded by angular closure. In this type of traverse there is no check on the systematic errors introduced into the measurement of lengths. Therefore, when this type of traverse is executed for a major project, the measuring apparatus must be carefully calibrated to determine the systematic errors and to eliminate them.

A traverse that originates a known position and closes on another known position is by far the most reliable, because a check on the position of the final point checks both the linear and angular measurements of the traverse.

Precise Traverse : Under certain conditions a traverse may be substituted for a triangulation work. In flat, heavily wooded country the construction of towers and signals is a large item of expense in establishing a triangulation system. When such an area is crossed by railroads and paved highways, traverses may replace the triangulation. In deciding between the triagulation and traverse, the following should be kept in mind. Traverse stations will be more available for the control of local surveys than will triangulation stations, since the latter are generally located in places more difficult to access. Because of this inaccessiblity, the transportation of materials for the erection of towers and signals may be impossible and thus preclude the use of triangulation. On the other hand, triangulation stations are usually on high ground, and therefore are visible from much larger areas than are traverse stations. Besides, an arc of triangulation covers a belt of country at least 15 to 20 km wide whereas the traverse is just a single line.

The electronic distance measuring systems are successfully employed in measuring the lengths of line in precise traverse. On shorter lines, taping will prove more economical.

As in the case of triangulation and trilateration, traverses are classified as first order, second order or third order, according to the degree of precision attained.

No check on the precision of the traverse is available until a traverse has been closed and the latitudes and departures and the closures have been computed. The discrepancy is sometimes of such size that it is difficult to tell whether it is caused by a blunder or by an accumulation of small errors in the traverse. When a blunder has been made, it is often difficult to locate the incorrect value and it may be necessary to rerun a large part or all of the traverse lines. In triangulation as each quadrilateral is completed in the field, there are checks on both the angles and the lengths. Furthermore, the office computations for the adjustment of triangulation have automatic mathematical checks that are lacking in the computations for a traverse.

Precision of Theodolite Travaersing : The angular error of closure in theodolite traversing is generally expressed as equal to $L\sqrt{N}$, where L is the least count in seconds of the instrument used which may vary from 1" to 1', N is the number of angles measured. The degrees of precision in angular and linear measurement in a traverse under different circumstances are given below :

Type of traverse	Angular error of closure	Total linear error of closure
(1) First order traverse for horizontal control	6" N	1 in 25,000
(2) Second order traverse for horizontal control and for important and accurate surveys	15" N	1 in 10,000
(3) Third order traverse for surveys of important boundaries, etc.	30" N	1 in 5,000
(4) Minor theodolite traverse for detailing	1' N	1 in 3,000
(5) Compass traverse	15' N	1 in 300 to 1 in 600

QUESTIONS & ANSWERS

Question 1. What are the classifications of the triangulation survey ? Describe each briefly.

Answer : CLASSIFICATION OF TRIANGULATION SURVEY

Triangulation system may be classified as follows according to the degree of accuracy desired and the magnitude of the work :-

1. Primary or first-order.
2. Secondary or second-order.
3. Tertiary or third-order.

1. **Primary triangulation :** It is the main triangulation of a country carried out with every care where the sides of the great triangles vary from about 30 km to 150 km or more, and the length of base line from 5 km to 15 km. In work of this character two series of chains of large and well-proportioned triangles are laid out, one along the meridian and the other approximately at right angles to the meridian. The areas enclosed are then covered with a net work of smaller triangles. In this system most refined instruments and methods of observations are used for most precise horizontal control for small-scale mapping surveys.

It is very slow and expensive. The degree of accuracy in primary triangulation is 1 in 500,000 and the check on the base is 1 in 25,000. An average triangular error should not exceed 3 seconds in such a survey.

2. **Secondary triangulation :** It breaks up the principal triangles of the primary triangulation into smaller sides of 8 km. to 60 km and carried out the form of secondary chains connecting the principal chains. The length of the base line in secondary triangulation varies from 2 to 5 km.

The degree of accuracy is 1 in 50,000 and the check on base is 1 in 10,000. A triangular error upto 8 seconds is allowed in this system.

3. **Tertiary triangulation :** Within the secondary triangles points are established at short intervals to break up the secondary triangles into sides of maximum upto 8 km. in length. It furnishes the horizontal control for detail surveys.

The degree of accuracy is more than 1 in 5,000 while the check on base is 1 in 5,000. A triangular error of 15 seconds is allowed in this system.

Question 2. Describe the principles involved in a triangulation survey. It is proposed to adopt triangulation method of surveying the surface of a mine, what are the points you would keep in mind while reconnoitering for fixing the position of stations ?

Answer : PRINCIPLES IN TRIANGULATION AND CONSIDERATIONS FOR FIXING STATIONS

Principle : On the surface, where the country is comparatively open, the survey is performed by triangulation, It is based on the trigonometrical proposition that if only one side and all the three angles of a triangle be known then the remaining sides can be calculated by the sine-rule. Suitable triangulation stations form the vertices of a series of mutually connected triangles of the triangulation system. In this system one side called the base line, and all the angles are very

carefully measured and the lengths of all other lines in the system are then computed. All the angles being known, the direction of each line relative to some standard direction may be determined, and thus the position of each triangulation station with reference to a predetermined point in the area surveyed can be computed by co-ordinates.

Considerations for Fixing Triangulation Stations

The following points should be considered for fixing the positions suitable for triangulation stations -

1. A careful study of all available surface plans of the area should be made for fixing probable positions of triangulation stations.
2. The station points should be so chosen that the stations of each triangle are intervisible. The highest available ground should be selected for intervisibility.
3. They should form well-conditioned triangles. A right angle is the angle least liable to error, but the error tends to increase slowly as the angle becomes more obtuse and more reapidly as it becomes more acute. For this reason the angle of every triangle should be as regular as possible, preferably not less than 30^0 and not more than 120^0. An isosceles triangle with angles at the base $56^015'$ each and that at the apex $67^030'$ is the ideal triangle.
4. The length of sides should be neither too short nor too long. For long sights accurate bisection of signals may not be possible and for short sights errors of centering may srise. For colliery triangulation it should be between 300 m to 500 m depending on configuration of the land, obstacles and the scale to be used for plotting.
5. The base line should be located on flat ground so that well-shaped triangles can be formed in connecting the end of stations of the base to the main triangulation stations.
6. All stations should be easily accessible.
7. They are so located that the cost of clearing obstructions from the line of sight is minimum.
8. They should be useful for detail surveys. Important surface features such as shafts, inclines, bunkers, important buildings, etc. should be located near the triangulation stations.
9. The number of main stations should be minimum.
10. Stations should not be located in an area under extraction to avoid any movement of the stations during the progress of survey.
11. Stations should be capable of being sighted at or near the bottom-most points to avoid errors due to non-verticality of signals.
12. They should be fixed on firm ground avoiding sand or loose earth.
13. Two inaccessible stations should not be used in any one triangle.
14. Height and convenience of access to station must be considered.
15. When jungle or undergrowth obstructs the line between the two stations, stations may be raised by erecting wooden scaffold so as to clear the greater part of the obstruction avoided and the remainder cleared at small cost.

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16. Stations should be permanently marked and sited by suitable measurements with reference to outstanding features and these should be recorded in the survey book.

Question 3. Describe how you would select and measure the base line for a colliery triangulation. What corrections would you apply to the measured length ?

Answer : SELECTION AND MEASUREMENT OF BASE LINE

Selection of Base Line : In a triangulation survey the base is of prime importance. Since the accuracy of the computed sides of the whole triangulation system fully depends upon the accuracy of measurement of the base line, utmost care should be taken in selecting the site, length and its measurement in the field. The following points should be considered in selecting the site for a base line :

1. The site selected for the base line should be fairly level, evenly sloping or gently undulating.
2. It should be free from obstruction throughout the whole of its length.
3. The two ends of the base line must be visible from each other and as well as from at least three other triangulation stations.
4. It should be located somewhere in the central portion of the area to be surveyed so that the net need not be extended too far in one direction.
5. If the area is greatly elongated, the base line should be located as far as possible in the direction of the longer dimension.
6. The ground should be firm and smoorth.
7. It should not be too long or too short.
8. The survey stations should be easily approachable.
9. The site should be so selected that well-conditioned triangles can be formed in connecting the end stations to the main triangulation stations.

Measurement of Base line : The following instruments are required to measure a base line :

1. Theodolite.
2. Dumpy level and levelling staff.
3. Standardised invar tapes 60 m long, one for fieldwork and one for comparison.
4. Spring balance and tension handle.
5. Wooden pegs, 8 cms. square section.
6. Zinc plates.
7. Steel bar, short length of brass chain, thermometers.
8. Nails, knife-blade, spade, grass-cutter and hammer.
9. Field book, pencil, plotting scale, set square, etc.

Procedure : In rough and undulating grounds or in hilly countries measurements are made off the ground by a steel tape hanging in catenary between two supports (boning rods) with additional intermediate supports.

In level or slightly inclined ground the line to be measured is divided into bays by fixing pegs on the ground aligned with a theodolite and grass and bushes cut so as to provide a smooth surface for steel tape to be placed over it for measurement.

A base line on fairly level and even ground maybe measured in the following way :

Greatest care must be taken in measuring a base line as any error in its measurement is multiplied throughout the whole work. The ends of the base line should be accurately fixed by concrete pillars. The line should be measured in equal bays which are slightly less in length than the length of the tape used.

A theodolite is carefully centred and levelled over the station at one end of the base line, and sighted on the station point at the other end. A number of wooden pegs capped with zinc plate are then ranged in with the theodolite in line and driven into the ground, the distance between the consecutive pegs being some what less than the length of the tape to be used.

The pegs driven should have no shake, and these should project above the surface of the ground so that when the tape is stretched under tension between the two pegs, the tape is suspended clear of the ground. Longitudinal lines are scratched on these plates using the theodolite to ensure that each scratch is truly in line.

The tape is stretched out to lie on the first bay length. The zero of the tape is connected by a short length of brass chain to a steel bar driven into the ground just ahead the starting station. When the tape is stretched out the length of the brass chain is so adjusted that the tape is fairly taut, its zero mark is a little behind the starting station and the forward end is a little ahead the draft peg. The spring balance attached to the tape end should indicate that the desired tension has been attained. Intermediate supports between the station and the draft peg can be given by means of additional pegs just touching the tape underneath.

The reading of the tape at the correct point of the starting station is then noted. A mark made with knife-blade on the zinc-plate on draft peg, and the reading of the tape at this mark is also noted.

The temperature of the tape for this draft is determined from the mean of the thermometer readings at three points equidistant from one another.

The procedure is carried out at least two times for this bay and the average taken as the measured length of the bay.

The tape is then carried forward and the same procedure is applied to successive bays the zero end of the tape being in each case applied to the knife-blade mark termination the en of the previous bay. When the whole of the base line has been so measured the measuremen should be operated in the reverse direction.

Flying levels are taken to determine the R.L. of each peg and the ends of the base lines b means of Dumpy level and staff. If the ground is inclined theodolite is used to measure the inclination.

The length of the tape used is compared with the second standardised tape kept for the

purpose after the fieldwork.

The total length of the base line between its two ends will be obtained by adding the consecutive bay lengths, necessary corrections are to be applied to the field measurements in order to get its true length.

Bay	Correction		Length of Bay	Temperature				Remarks
	+ ve	– ve		1	2	3	Mean	
A	–	–	–					To B
1								Standard
2								tension applied
								"
								"
B								"
A to B								

Corrections :

The following corrections are to be applied to the field measurements of a base line in order to get its true length -

1. Correction for standard or abstract length.
2. Correction for horizontal alignment
3. Correction for slope or vertical alignment.
4. Correction for sag.
5. Correction of temperature.
6. Correction for pull or tension.
7. Reduction to sea level.

It should be noted that each section of the base line is separately corrected. If the standard tension is applied to the tape and the base line is measured in a straight line, correction for tension and horizontal alignment are not applied.

Question 4. A base line selected on an undulating ground in a colliery site as a last resource. Exaplain the arrangements which you would make in order to measure such a base line.

What corrections will be needed in order to compute the length with requisite acuracy ?

Answer : MEASUREMENT OF BASE LINE WITH TAPE IN CATENARY

In rough and undulating countries measurements are made off the ground by an invar steel tape hanging in catenary between two supports with additional intermediate supports.

Instruments Required :

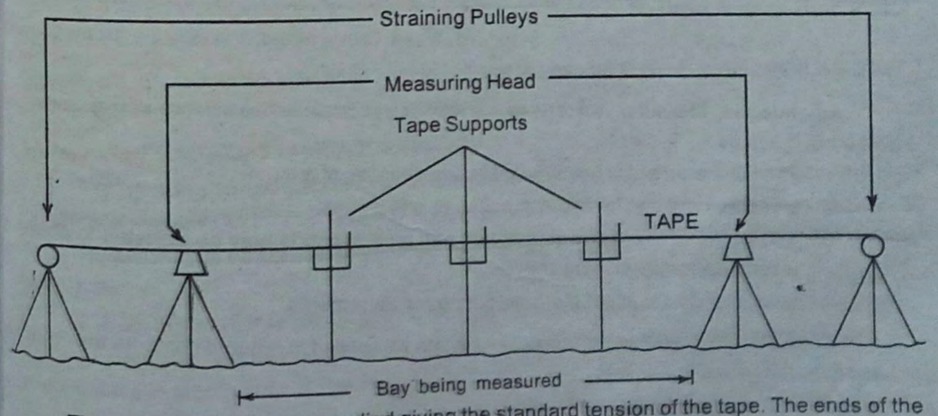
1. Theodolite, and levelling staff.
2. Two Standardised invar tape, each 100 m long.
3. Two self aligning pulleys mounted on tripod incorporated with straining device.

4. Two measuring heads mounted in a tribrach screwed on to a tripod.
5. Transferring head.
6. Tape supports with freely mounted pulleys and clamps which can slide on wooden poles.
7. Sighting vane.
8. Optical plummet.
9. Thermometers.
10. Wooden poles.
11. Field book, pencil, wooden plugs, etc.

Procedure : The base line to be measured is cleared of obstructions such as trees, bushes, etc. and its first bay is approximately set out. The ends of the base line are accurately fixed by concrete pillars.

The first measuring tripod is set over one end of the base line and accurately centred over the reference mark using the optical plummet. The theodolite is set over the measuring head and the far end of the base line is sighted. The second measuring head is set in line at the end of first bay length slightly less than one tape length. Tape supports are placed in line at proper intervals, the points of support being set either at the same level or on a unifrom gradient. The two straining tripods are then set about one metre back on either side of the ends of the bay to be measured. The pulleys of these tripods have screws to allow tansverse movement to ensure correct alignment of the tape.

The tape is set in position, ensuring that there is no twisting. The tape is now suspended between the straining pulleys and lies on the measuring heads and tape supports. Slight difference in level between the ends of the bay being measured may be adjusted by setting the tripods and tape supports so that the tape remains level. If the change of level is too great for this, the inclination of the tape must be obtained to reduce the measured length to the horizontal.



The straining weights are applied giving the standard tension of the tape. The ends of the tape are read simultaneously through the magnifying eye-piece of the measureing heads. The

temperature of the tape is determined by three thermometers one placed near each end and near the middle of the tape.

The tape is moved a short distance one way and two further readings recorded. It is then moved a short distance in opposite direction and further readings taken.

To mark the ends of the bay the measuring head is substituted by a transferring head into the tribrach and the exact position of the measuring head is marked by a plug in the ground by means of the optical plummet.

The remaining bays of the base of the base line are measured in a similar way, the leading measuring head of a bay acting as the back station of the next bay. When the whole of the base line has been so measured the measurement should be repeated in the reverse direction. After the day's work the field tape should be tested with the second tape kept for the purpose.

Necessary corrections are to be applied to the measured length of each bay separately to determine the true length of the bay. The only disadvantage of this method is that strong wind may cause sway of the tape and affect the accuracy of the work.

Field Book.

From	To	Face	Distance in metres	Vertical Angle A ° ' "	Vertical Angle B ° ' "	Mean ° ' "	Mean ° ' "	Slope	Hor.Dist. in metres	Relative Altitude	
A											210.000
	1	R	86-60	0-9-30	0-10-00	0-9-45	0-9-38	Rising		0.240	210.240
		L		0-10-00	0-9-00	0-9-30					

Remarks : Tension, Standard, Mean Temp. noted and Bay length measured 3 times – Mean noted.

Advantages : Measurement of base line with tape in catenary has the following advantages over other methods :-

1. The measurement is not affected by unevenness of ground.
2. Measurement can be taken in bushy or odd places.
3. The height of the measuring heads and tape supports may be adjusted to obtain a level or uniformly sloping line.
4. Mud, etc. do not affect the tape during measurements.

Correction : The following corrections are to be applied in order to compute the length of base line with requisite accuracy :-

1. Correction for standard length.
2. Correction for slope.

3. Correction for sag.
4. Correction for temperature.
5. Reduction to sea level.

As the base line is set out in one straight line and the standard tension has been applied to the tape, corrections for horizontal alignment and that for tension will not be needed.

Question 5. Under what conditions Base Extensions become necessary in triangulation ? What are its advantages ?

Answer : EXTENSION OF BASE-LINE AND ITS ADVANTAGES

The measurement of a long base-line may some times be difficult due to obstacles or the level ground being of limited extent. In such cases, only a portion of the base line accurately measured which is then prolonged by ranging lines in continuation of it, at one or both ends, until a suitable length is obtained to give well-conditioned angles from each of its ends to the points to be fixed. The lengths of the additional lines are calculated from the measured base and the observed angles.

Advantages : The following are the *Advantages of Base Extension* :

1. Measurement of a longer base may be impracticable due to obstacles which may involve a considerable amount of clearing and preparation of the ground selected as a base. It is therefore always better to measure a short base line very accurately and extend it by employing some trigonometrical method such as *Snellius Rhombus method or by Double Extension.*

2. Linear measurements are adversely affected by poor topographical conditions. The linear measurements for a longer base are generally affected by constant errors. These may be avoided by selecting a site for a shorter base on a more or less level ground in the area.

3. The amount of cutting and clearing necessary to make the line of sight free from obstruction for a shorter base line is greatly reduced. Time, labour and cost are also saved.

4. A short base line may be measured with highest degree of accuracy and necessary corrections may also be applied.

Question 6. Explain the precautions, which you would take in the measurements of angles and base lines in order to attain accuracy in triangulation surveying.

Answer : PRECAUTIONS IN MEASURING ANGLES

The following precautions should be taken in measuring angles in order to attain accuracy in triangulation survey :

1. To eliminate errors in adjustment of the instrument observations should be repeated on both faces.

2. Both the vernier readings should be taken to eliminate errors due to eccentricity.

3. Angles should be read on different parts of the instruments to avoid errors arising from irregularities in graduations of the limbs.

4. The upper part of an elevated station is, in clear weather, usually subject to a twisting motion in the direction of the sun's motion. Precautions should be taken to eliminate this error by

observing angles in both directions–by 'swing right' and 'swing left'. This precaution is also considered to equalise the movements arising from exceedingly small slip of the clamps or 'give' at the levelling head or tripod.

5. Precautions against backlash should always be taken in the use of micrometer screws.

6. Refraction always introduces an element of uncertainty, and lateral refraction is more disconcerting than refraction in altitude. Sights should be kept well above the ground. Also observations should be taken in densely clouded weather or from the late afternoon to sunset.

7. An error may arise due to inaccurate bisection of the object being sighted. Bright sunlight on one face with dark on the opposite face of the signal may lead to an error. For accurate work with short sights a plumb bob suspended over the station-point should be sighted.

8. Care should be taken that the ranging rod is placed vertically in the correct line of the station. The lowest point visible should be sighted.

9. When plumb-bob is used for sighting purpose care should be taken that it is not deflected to wind or other causes.

10. Parallax should be eliminated.

11. Observations in mist or stromy weather should be avoided.

12. In addition to above mentioned precautions, every care should be taken to guard against possible mistakes due to -

(a) Confusion and observation of wrong station-points.

(b) Manipulation of wrong tangent screw.

(c) Wrong application of an index error–positive instead of negative, or vice versa.

(d) Reading of the wrong vernier.

(e) Omission of a half degree when the primary scale is so divided, e.g. 112°15' for 112°45'

(f) Incorrect reading of the scales.

(g) Incorrect booking.

A systematic observations by method of repetition should be followed which tend to eliminate personal errors in the bisection of object, the reading of verniers, etc.

Precautions in base-line Measurement.

1. The reference tape should be perfectly checked.
2. Errors in deducing the length of the field tape from the reference tape should be guarded against.
3. Correct pull in a tape should be recorded and used for pull correction.
4. Error in the determination of the weight and cross-section of a tape, affecting corrections for sag and pull should be guarded against.
5. Care should be taken against creeping error due to incorrect determination of temperature.
6. Precaution should be taken to determine the co-efficient of expansion of various metal used.

7. Errors in alignment should be guarded against.
8. Error in the determination of the inclination of line should not keep in.
9. Error in the reduction to mean sea level should not intrude.
10. Personal errors in reading scales, bisecting and marking should be taken care of.
11. Displacement of the marking pegs in tape measurement should not occur. Straining peg should always be provided.

Question 7. Enumerate the sequence of the operations necessary before the measurement of the base line commences.

Answer : SEQUENCE OF OPERATIONS BEFORE BASE-LINE MEASUREMENT

The sequence of operation necessary before the measurement of the base line are -

1. To select a suitable base line, and to make the line of sight free from obstruction.
2. To fix the end of the base line, by concrete pillars.
3. To divide the base line with equal bays which are slightly less than the length of the tape used.
4. To fix wooden pegs capped with soft zinc plates in ground at the ends of each bay after correct alignment over the base line with the help of a theodolite.
5. To determine reduced level of pegs by means of a dumpy level and staff.
6. To test the field tape with a standard tape kept as reference.
7. To determine the weight and cross-section of the field tape.

Question 8. Discuss in detail the relative merits of triangulation and precise traversing for carrying out the survey of an extensive area.

Answer : TRIANGULATION AND PRECISE TRAVERSING

Triangulation : On the surface, where the country is comparatively open, the survey is performed by triangulation. This consists of setting forth a series of triangles from a main base line to comprehend the whole of the area to be surveyed. Providing the length of the base line and the angles of the triangles are very carefully measured and a proper system of check lines run, then, for accuracy, triangulation leaves nothing to be desired.

The direct object of triangulation is not to provide a complete plan of the area but to locate a number of points over the surface of the country very precisely which form as base for further interfilling by subsidiary surveys. The degree of accuracy for fixing these points depends upon a number of factors such as extent of survey and scale to which final plan is to be made, instrument used, and time and funds available and also the purpose of the map. Triangulation survey is employed for major survey work for entire country.

In the mines, underground workings should be accurately shown on plan with reference to surface survey, and as such surface points should be precisely and accurately established only possible by triangulation.

Since in triangulation it is only required to determine the length of the base line every

possible refinement can be used to obtain accuracy. Angular measurements are not affected by constant errors as in linear measurements and therefore angles can be determined with the highest degree of accuracy.

Precise Traversing : A precise traverse survey consists of a continuous series of lines from which offsets are taken in usual manner. The relative directions of these lines are determined by angle measuring instruments.

The accuracy in precise traversing depends upon the degree of refinement of instruments and method of observation, and the purpose of the survey. In mine surveying we must be content to follow the more or less tortuous roadways taking the bearing of each irregularity with reference to magnetic meridian or the angle formed with the portion immediately preceeding or suceeding. In polygonally closed traverse a complete circuit is surveyed and survey may be checked or adjusted.

Precise traverse, like triangulation surveys, are also used to find the accurate position of relatively few marked points or stations with less precise measurements to features to be mapped or located, without accumulating accidental errors. These usually serve as control surveys and are cheaper and more effective in small areas or where many obstacles interfere with sight lines.

Question 9. What are three methods used in traversing for direct observation of bearings. Describe any one method and its advantages.

Answer : DIRECT OBSERVATION OF BEARINGS

There are three methods used in traversing by the FAST NEEDLE METHOD for observation of bearings directly in the field :

1. **Direct Method by Transiting the Telescope -** In this case the deflection angle is automatically added to the bearing of the preceding line.

2. **Direct Method without Transiting the Telescope -** In this method the exterior angle (in case of clockwise traverse) is added to the bearing of the preceding line automatically and the 180^0 correction is to be made.

3. **Back Bearing Method -** In this method the vernier reading is set to the back bearing of the preceding line at each station for orientation before a back sight is taken on the preceding station. The telescope is not transited but simply rotated horizontally for taking foresight on the forward station.

DIRECT METHOD WITHOUT TRANSITING TELESCOPE

After performing temporary adjustments of the theodolite at station A, the bearing of the line AB is noted. The theodolite is next shifted and set up at B. The backsight is taken on A with the vernier reading the bearing of AB exactly. The telescope is rotated in a horizontal plane to bisect C. The orientation at B being 180^0 out, a correction of $\pm 180^0$ has to be applied to the reading of the vernier taken at station B. At the next station the orientation is 360^0 out and is therefore correct and no need to apply the correction of 180^0. The correction is required only at even instrument stations.

Advantage :

1. The method is suitable in case of non-transit instrument.
2. This is preferred in the case of transit instrument when it is in poor adjustment.
3. In point of accuracy, it is satisfactory.
4. Errors due to imperfect adjustment of the line of collimation are not introduced at any station as in the case of direct method by transiting the telescope.

Question 10. Write short notes on -

(a) Satellite Station, (b) Broken base-line.

Answer : (a) SATELLITE STATION

It sometimes happens, when church spires, temples, towers or other remarkable objects are selected as stations, for the continuation of triangulation that the theodolite can not be placed over the point occupied by the axis of the signal or the rays from the station are obstructed or a scaffold may have to be erected for the purpose.

In all such cases, the necessary observations may be taken from some other convenient station a short distance away, and this station is known as a Satellite of Supplementary Station. The observations therefrom can be adjusted to agree with these which would have been made from the main triangulation station. With best laid out triangulation it is never used and it can be easily avoided with little care.

(b) BROKEN BASE LINE

It may not be sometimes possible to measure the whole length of a base line because the intermediate length of it contains of obstruction. In this case, the base may be lined out and the two lengths on either side of the obstruction measured in the usual way. To obtain the length of the intermediate section a theodolite is set up at a suitable point so that all the four ends of the two length on either side of the obstruction can be observed and all the angles are measured at the station and finally the required length is calculated trigonometrically.

Let AB (a) and CD (b) be the portions of the base which are measured directly, and BC (x) the portion which can not be measured directly. A suitable station E is selected and the angle AEB (α) AEC (β) and AED (δ) are measured.

From the triangles AEC and AEB,

$$\frac{CE}{BE} = \frac{(a + x)\ \text{Sin}\ \alpha}{a\ \text{Sin}\ \beta}$$

Again, from the triangles CED and BED.

$$\frac{CE}{BE} = \frac{b\ \text{Sin}\ (\delta - \alpha)}{(b + x)\ \text{Sin}\ (\delta - \beta)}$$

whence, $(a + x)(b + x) = \dfrac{(ab\ \text{Sin}\ \beta)\ \text{Sin}\ (\delta - \alpha)}{\text{Sin}\ \alpha\ \text{sin}\ (\delta - \beta)}$

$$\therefore x = + \sqrt{\left\{\frac{ab\ \sin\beta\ \sin(\delta - \alpha)}{\sin\alpha\ \sin(\delta - \beta)} + \left(\frac{a - b}{2}\right)^2\right\}} - \frac{1}{2}(a + b)$$

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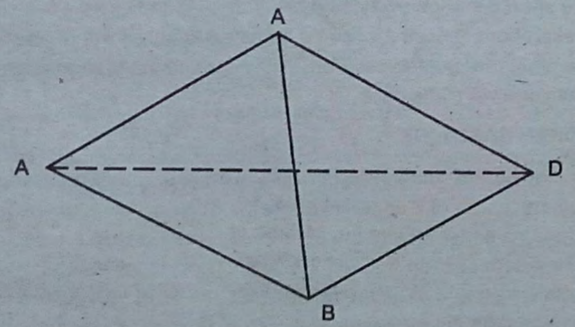
Question 11. What is the usefulness of a 'Snellius Rhombus' in triangulation survey ? Explain how you will include such a Rhombus in the scheme and solve the problem of obstacles in base lines. What should be the limiting value of the apical angles of the Rhombus and why ?

Answer : SNELLIUS RHOMBUS

Length and position of a triangulation base is generally determined by the area and nature of the ground to be surveyed. The longer the base the easier it will be to build up a triangulation comprising a comparatively small number of well-shaped triangles. To obtain a small number of well-shaped triangles a base of sufficient length may be selected, but a considerable amount of cleaning and preparation of the ground selected as a base may have to be done in such cases. Measurement of longer base may be impracticable due to obstacles and the accuracy of linear measurement may be adversely affected by poor topographical conditions. To solve the problem of obstacles, it is therefore better to measure a short base line very accurately and extended it by employing some trigonometrical method. This system of base extension is done by employing the Snellius Rhombus.

In the figure if AB represents the short measured base, two triangles, ACB and ADB, are laid down. The figure ADBC is known as the Snellius Rhombus. If the base AB and the three angles of each triangle are measured, the length of CD can then be calculated which will serve as a longer base on which the whole triangulation can be built up. The shapes of the two triangles ACB and ADB must be considered when adopting this system of base extension. If such triangle is equilateral, the length of the extended base CD will be 1.732 times the measured base.



Snellius Rhombus

Triangles with small apical angles will increase the rate of extension, but the error in the calculated length of the extended base may be too great although care is exercised in the measurement of the small angles.

When measuring the angles of the Snellius Rhombus smaller angle should be measured more precisely than the larger ones, the measurement should be so arranged that the effects of the average errors of all the angles are equal to one another.

Permissible average error of the measurement of each angle can be calculated from the formula -

$$a = \pm 146{,}000\, f \tan A,$$

Where, $\pm a$ = Average permissible error in seconds in measurement of the angles under consideration.

$\pm f$ = Desired average fractional error in the calculated length of a side of the Rhombus which should not exceed $\frac{1}{10{,}000}$.

A = Angle under consideration.

For example, if magnitude of each apical angles is 20^0, the permissible average error in the measurement of each apical angle will be $\pm$ 5.5 seconds when the desired average error in calculated length of any side of rhombus is $\pm\frac{1}{10{,}000}$ and that of the extended base is $\pm\frac{1}{11{,}500}$. The extension of base in such case will be 11.430 times.

The apical angles of the triangles of a Snellius Rhombus should not be less than 20^0 to keep the average error in the measurement of the apical angles within a limit.

Question 12. Describe how you will determine the true north -

(a) By observation of the Pole Star.

(b) By shadow method.

(c) By a watch.

Answer : TRUE NORTH DETERMINATION - APPROXIMATE METHODS

By observation of the Pole Star :

In the Ephimeris are given the position of 35 Circumpolar Stars, several of which are suitable for observations in determining the True North in the Northern Hemisphere. But the polaris or the pole star is the only one, however, that is bright enough to be seen easily with the naked eye.

The Pole Star describes a small circle about the invisible pole once each sidereal day. Twice each day the star is on the meridian; once when directly above the pole, called 'upper culmination' and again when below the pole at 'lower culmination'. Its extreme eastern position is the 'eastern elongation' and its extreme western position is called the 'western elongation'.

The Pole Star is bright star which forms the first star in the tail of circumpolar constellation known as Ursa Minor or Little Bear of Little Dipper. To identify the pole Star, first the Ursa Major or Great Bear of Great Dipper, comprising 7 fairly bright stars forming a dipper with bent handle is to be found out. The two stars on side farthest from the handle are called the Pointers, because a line through them, prolonged about 5 times the distance between them, passes almost through the Pole Star.

Now a fine plumb-line is set up at a place where it is not likely to be disturbed by air. The observer placing himself behind the plumb-line will find a moment when the Pole star and the Alioth (the third star of the bent tail of the Great Bear) are in a line with the thread of the plumb-line. The direction marked on the ground will then be in the meridian pointing towards the true north from the observing station. This method is approximate.

(b) By Shadow method :

The method is the most common of all those employed in determining the direction of the true north, and it is based on the fact that the sun casts shadows of equal length at equal intervals of time before and after passing across the meridian. Thus if a staff be placed vertically on level ground and direction of shadow be marked at a certain time before noon, and again after lapse of an equal period of time after noon, the direction of the true meridian will be found by bisecting the angle between the directions of the shadow before and after noon. Such a method although giving fairly reliable results, is correct only at the solstices, that is on December 22 and June 21.

Concentric circles of varying radii are drawn round the centre on a piece of paper fixed to a level table (or a plane table after levelling).

A straight steel pin about 0.3 cm thick and 12 cm long is planted at the common centres of the circles. The verticality of the pin is checked, in four directions at right angles to each other, either with a right angle of a set square or by a plumb bob. The measurements are started in the morning and completed in the evening between 8 A.M. and 4 P.M. as the top of the shadow cast by the pin, just touches a particular concentric circle, the point of contact is marked on the circle by a cross. Thus at the end of the day, when all such crosses are joined, two lines meeting at the centre of the circles are obtained. The magnitude of the angle between the two lines, depends on the latitude of the place and also on the season of the year. The bisector of this angle will give the direction of the true north.

(c) By Watch :

A watch is held so that the hour hand points to the sun, and the angle formed between the hour hand and the '12' mark on the dial is bisected. If the bisecting line is prolonged backward it will indicate the direction of the true north.

It is also possible in this way to tell the angle between the true meridian and a certain survey line, for if the difference in minutes (time) between the direction of the meridian and the direction of the survey line be multiplied by six, the result will give approximately the number of degrees in the angle required.

Question 13. Enemerate the various mothods of determining the true meridian. Describe one method which has the necessary accuracy for an important triangulation survey.

Answer : METHODS OF DETERMINING TRUE MERIDIAN

True meridian : The true meridian or geographical meridian through any point on the earth's surface is the imaginary vertical plane which passes through that point of observation and also through the North and South geographical poles. The direction of true meridian for any gien place is always constant.

Methods : There are several methods for determining the true meridian at any place; though astronomical observations are correct.

(A) Approximate Methods :

1. Direction of shadow at apparent noon.
2. Magnetic Compass.
3. Circumpolar star on the same vertical.

4. Observation of Polaris or other Circumpolar star.
5. Survey Maps.
6. Watch.

(B) Accurate methods :

1. Equal altitude of the Sun.
2. Equal altitude of circumpolar star.
3. Extra-meridian observation of the Sun.
4. Extre-meridian observation of a circumpolar star.
5. Two greatest elongation of a circumpolar star.
6. Single observation of a circumpolar star at elongation.
7. Gyro-Theodolite.

METHOD : Observation of a Circumpolar Star at Equal Altitudes

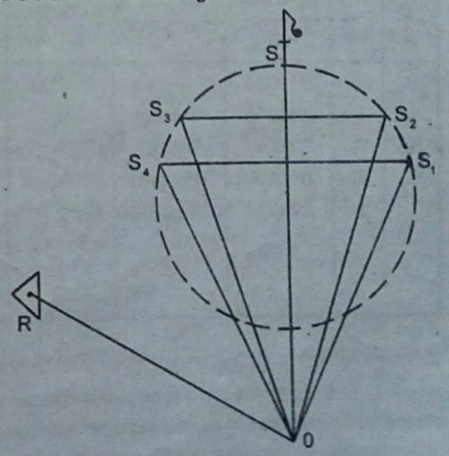
For finding true meridian or true north, the observation of a circumpolar star at equal altitudes is preferred in colliery surveying owing to the method being simple and accurate, and independent of latitude and time.

Equipment Required :

1. A transit theodolite.
2. A good watch.
3. Lamp for illuminating Reference Station, and a reflector to throw light into the telescope field and make cross-hair visible.
4. Field book, pencil, drawing instruments, sighting signal, torch, etc.

Principle : If a circumpolar star is sighted from point O at a certain altitude when east of meridian and again sighted at the same altitude when west of meridian, the line bisecting the horizontal angle formed by the two positions of the star with observing station is the true North meridian.

Procedure : Assuming that station 'O' has already been permanently marked out and its position fixed with regard to other survey station, the theodolite is set up at station O and centred and levelled accurately by means of telescope bubble tube. The vernier is set to Zero and the two horizontal scale verniers noted. The body clamp is released and the reference station R which has been permanently fixed is accurately bisected. The telescope is then directed towards the probable direction of the star by clamping the body and unclamping the vernier. After the star has been accurately bisected, at S_1, verniers A and B on horizontal circle for angle ROS_1 and also on vertical circle α, are read and booked. Exact time of observation is also noted.



By Equal Altitudes of the Sun

The face of the instrument is changed, the telescope redirected to the reference station R, both verniers read, and after 15 to 20 minutes, the same star at S_2 is again bisected with cross-hairs and horizontal angles ROS_2 and the vertical angle α_2 from both verniers recorded. The time of bisection is noted.

The theodolite is kept at the same station O till the star goes to the west horizon. The telescope with the same altitude α_2 as at S_2 is directed to the star in position S_3 and after exact bisection the horizontal angle with the reference station, ROS_3 is observed on both verniers and exact time of bisection noted.

The face of the instrument is changed to its original face, the reference station R intersected, the vertical circle vernier is adjusted to read α_1 and the star observed in the telescope until it coincides exactly with the intersection of the cross-hairs, when the horizontal angle ROS_4 on both verniers and the time are recorded.

The line bisecting the horizontal angles $S_1 OS_4$ or $S_2 OS_3$ is the direction of true meridian. The values of the vertical angles are not used further. The angle ROS is computed as the mean of the eight readings of the horizontal angles to S_1, S_2, S_3 and S_4. The true North line can then be easily laid out on the ground with the help of the reference station as the angle ROS in known

Observation Table

Observation for True Meridian ..... By .....(Star) at Equal Altd.

Place ..... Colliery, Date ..... 16.10.1997.

By ..... (Instrument) Time .....

Referring Object Stn.R. of base OR, Station .....O, on of triangulation.

By ..... (Surveyor)

OBJECT	Face of instr.	Terms of Observn. h. m. s.	Elongation	Vert. Angle	HOR. ANGLE		REMARKS
					A	B	
R	L	210000	–	0°00'00"	0°00'00"	180°00'00"	Sky clear and dark
S_1	L	215000	Eastern	α_1			"
S_2	R	221000	Eastern	α_2			"
S_3	R	015000	Western	α_2			"
S_4	L	021000	Western	α_1			"
							All positions of star are on same side of ref. line, OR

Precautions :

1. The observation to the reference station should be made while it is still day and every care should be taken that the instrument left undisturbed.

2. A fairly bright star such as 'Sky King' is selected for obnservation on a clear and dark night.

3. The star selected should be such that it can be sighted two hourse before clumination.

and two hourse after culmination. The star should be indentified beyond all chance of errors, noting it on a diagram for certalnity some hours later.

4. The observations for different altitude should be made with different face of the instrument to avoid errors of adjustment.

5. At least four observatios should be taken in order to eliminate instrumental errors.

6. Both verniers should be read to avoid eccentricity error.

7. The altitude should be maintained precisely equal for each set of observation, i.e. for position of star at S_1 and S_4 or S_2 and S_3.

8. The altitudes α_1 and α_2 should not be very small, as refraction is then too certain.

9. The time interval between two sets of observation should be as long as possible, so that the star having a considerable velocity in altitude, i.e. shall not be very near its culminating points where the curve appears very flat, and a small change in altitude causes considerable displacement in azimuth.

10. Observations should be repeated two or three nights and mean of the results should be taken.

11. Observations should be made with greatest care. Proper attention should be paid to accurate bisection of the star, correct handling of the instrument and reading and booking angles, etc.

Advantages :

1. The method of determining the true meridian by observation of a circumpolar star is very simple and accurate.

2. The method requires no reflecting eye-piece.

3. Mathematical tables are not required for any calculation.

4. Calculations of local mean time, standard time, sidereal time or astronomical or civil day are not required.

5. It does not depend on the date, nor any reckoning of culmination, hour-angle, elongation or polar azimuth.

6. The sun's declination or the atmospheric refraction does not enter at all into the calculation.

7. Should there by any unsuspected error in the graduation or setting of the vertical arc, or some defect of the collimation in the telescope it would equally affect both part of the observer's work, and produce no effect in the resulting meridian.

Question 14. Describe, in detail, a good method of determining the true meridian in day time.

Answer : DETERMINATION OF TRUE MERIDIAN IN DAYTIME

The most common method of determining the true meridian in day time is by Equal Altitude of the Sun. This method of laying out the meridian is just as accurate as those which require the observation of the Pole Star at its extreme elongation, or at the time of its meridian passage, but a great deal more time must be spent in making the necessary observations.

Equipment Required

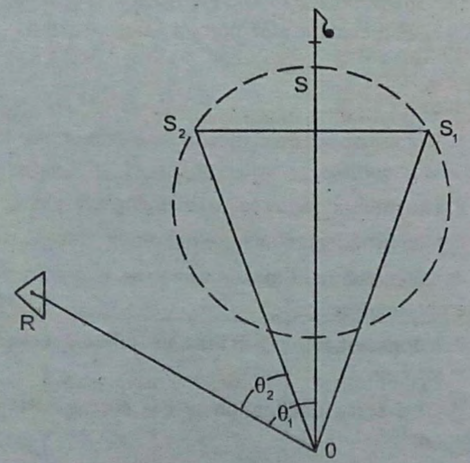
1. A transit theodolite with reflecting eye-piece.
2. A good watch.
3. Sighting signals
4. Field book, pencil, etc.

Procedure : The theodolite is set over a survey station O, and after the preliminary adjustments have been effected both plates clamped at zero, and with the vertical circle on left, the telescope is directed to the sighting signal at the reference mark R which has been permanently fixed and the station mark is accurately bisected.

The body of the instrument is clamped in that position, and the verniers are unclamped.

Several hours before noon the theodolite is turned horizontally, and the telescope is moved vertically until the sun is observed in the lower left-hand quadrant of the diaphragm with the cross-hairs tangent to it. The telescope is clamped in that position and the horizontal angle ROS_1 is noted from both the verniers. The time of observationis also noted.

The sun continues to rise and moves westwards till noon when it falls. When it reaches about the same altitude as before noon the limb of the instrument is turned horizontally till the sun observed at S_2 in the upper right-hand quadrant with the cross-hairs tangent to it. The horizontal limb is clamped in that position and the vernier readings are noted for the angle ROS_2. The actual time of the observation is also recorded.



By Equal Altitudes of the Sun

Let, the angle between the referring object and the sun at the time of first observation, $ROS_1 = \theta_1$

And the angle between the referring object and sun at the time of the final observation, $ROS_2 = \theta_2$

(i) When the two positions of the sun are on the same side of the reference line as shown in the figure, the angle between the referring station and the meridian,

$$\angle ROS = \frac{\theta_1 + \theta_2}{2}$$

(ii) But when these are on opposite side of the line, the angle between the reference station and the meridian

$$= \frac{\theta_2 + \theta_1}{2}$$

Owing to the change in the sun's declination during the interval between the morning and afternoon observations, the azimuth of the line so obtained requires correcting.

After necessary correction, the verniers are adjusted to the required angle and the meridian as set out by directing the telescope to at least two plumb-lines suspended at some distance from the instrument. It is desirable that the true meridian should be permanently established so that, at any time, the magnetic declination may be determined. The best method of marking the meridian is by means of cement concrete blocks under the surface of the ground having metal plugs inserted in them. The direction of meridian is defined by the line passing through the marks which may be made on the top of the metal plugs by means of a centre punch.

Precautions :

1. In solar observations certain times of day are more favourable than others, while at certain times the observations should not be made at all. During summer (In northern hemisphere) observations may be made at almost any time between 8 and 10 A.M. and between 2 to 4 P.M. Observations before 8 A.M. or after 4 P.M. are usually unsatisfactory, because the sun is so near the horizon that atmospheric refraction introduces a large error into the measured altitude of the sun; on account of refraction the sun appears to be higher than it really is, and the nearer the sun to the horizon the greater will be the refraction. For best results the sun should be at least 20^0 above the horizon. Between 10 A.M. and 1 P.M. the astronomical conditions are not favourable for definite results.

2. The reference mark should be far away to obviate the necessity of refocussing telescope in bisecting the mark after bisecting the sun.

3. It should be so situated that the line of sight is well above the ground to minimise the error due to lateral refractions.

4. A dark glass must be employed for the sun observation.

5. The observations for different altitudes should be made with different face of the instrument to avoid error of adjustment.

6. At least four observations should be taken in order to eliminate instrumental errors.

7. It is good practice, after observation is made, to sight back at the reference mark; if the vernier reads 0^0, it indicates that the horizontal circle did not move during observation.

8. It is undesirable to let the observations extend over more than 10 minutes of time, since the curvature of the sun's path will introduce error into itself for particular reading.

9. Observations should be repeated two or three days and the mean of the results should be taken.

10. Since the sun's centre cannot be bisected, observations should be made on the right-hand and left-hand limbs of the sun with telescope normal and inverted in both morning and afternoon observations.

11. Since the actual altitude of the sun is not required, the upper or the lower limb should be observed throughout.

12. The number or forenoon observations should be equal to the number of afternoon observations.

13. The time of each observation must be noted so that allowance may be made in calculation for change in sun's declination.

14. The azimuth of the line so obtained be corrected for any change in the sun's declination during the interval between the morning and afternoon observations.

15. All observations should be made with greatest care, and proper attention should be paid to the correct handling of the instrument and to the reading and booking of the angles.

[**Note** : Prismatic eye-pieces with coloured shades are furnished with transits for use in solar observations. If no such eye-piece is at hand and the telescope has an erecting eye-piece, the following procedure may be adopted :–

The telescope is pointed approximately at the sun, a white card board is held 8 cm – 10 cm behind the eye-piece, and the latter is drawn out until the image of the sun and the cross-hairs are both sharply focused on the card. The image may then be brought tangent to cross-wires almost as accurately as if the observer were looking through the telescope. If, however, the sun is high, the limb of the transit may interfere with holding the card in this manner. If the eye-piece is nonerecting the card–method cannot be used and hence a prismatic eye–piece is essential].

Question 15. How do you determine the latitude and longitude of a survey station ?

Answer : METHOD OF DETERMINING LATITUDE AND LONGITUDE OF A SURVEY STATION

Latitude : Latitude of any point on the earth's surface is the angular distance measured from the equator towards the nearer pole along the meridian at that point. The latitude of a point on the equator is 0^0, while at North and South pole latitude is 90^0N and 90^0S respectively and other points lie within these limits. Latitude north of equator is positive but south of equator is negative.

Latitude of a place can be determined by one of the following methods–

1. Altitude of Polaris at upper and lower transits.
2. Meridian altitude of a star or the sun.
3. Extra meridian altitude of a star or the sun.
4. Extra - meridian altitude of Polaris.
5. Circum - meridian altitudes of the sun or star.
6. Meridian altitudes of two stars.

To determine the latitude of a survey station by the Meridian altitude of the sun –

In this method the altitude of the lower or upper limb of the sun is observed with a transit theodolite when the sun is on the meridian. The true altitude is obtained after necessary corrections for index error, refraction, parallax and semi-diameter. Sun's declination at the time of observation is obtained from the known longitude of the place. Knowing the true-altitude and the declination of the sun, the latitude of the survey station can be calculated.

Latitude = (90^0 – altitude of the sun) + declination of the sun.

= Zenith distance of the sun + declination of the sun.

Declination is positive or negative according to the sun is north or south of the equator.

Solar Attachment : For determining the latitude a Solar Attachment is fitted to an ordinary transit theodolite which enables to solve astronimical triangle mechanically. Its polar axis is fixed to the centre of the telescope tube and at right angle to the line of collimation. A declination arc attached to the vernier revolves about the polar axis and covers an 'hour circle' which is read by a fixed index mark. Two horizontal lines (equatorial lines) and two vertical lines (hour lines) are ruled upon two small silver plates, one at each end of a rotating or radial arm, to facilitate for centering the image of the sun formed by the two lenses of the radial arm.

To determine the latitude, the theodolite with its solar attachment is set up and levelled accurately. The value of the sun's declination at apparent noon corrected for refraction is set on the declination arc.

Shortly before noon the telescope is directed due north so that image travels between the equatorial lines. The instrument is then clamped.

The declination arc then turned until it is parallel to the telescope.The image of the sun is kept within the small square of the solar screen adjusting vertically by the vertical tangent screw and horizontally by the tangent screw of the hour circle.

When the image ceases to fall below the lower equatorial line, it is apparent noon, and the index of the hour circle would then indicate XII.

The co-latitude or zenith distance of the sun is obtained from the vernier of the vertical circle of theodolite.

The required Latitude = 90^0 – (Co-latitude)

or, Complement of the angle read on vertical arc.

The results obtained by the use of the device are within one minute of truth.

Longitude :– Longitude of a place is the angular distance which the meridian of the place makes with some standard meridian such as Greenwich meridian taken as the zero. It is measured from 0^0 to 180^0 either towards the east or towards the west. It may be found by noting the difference between the local time and corresponding mean time of Greenwich at the same instant. For each 1 minute difference in longitude the difference in time is 4 secongs.

The following methods are generally employed for the determination of the longitude at a place –

1. By triangulation.
2. By chronometer.
3. By electric telegraph.
4. By wireless time signals.

To Determine the Longitude of a Survey Station By an Electric Telegraph :

The difference in longitude of two places may be determined by an electric telegraph connecting the two places. If the longitude of one place is known the other can be correctly ascertained.

Let, A and B are two places connected by electric telegraph, and A is to the east of B. A telegraphic signal is sent out at A which is received at B.

2014:01:01 07:21:48

t_1 = time at A when signal sent out at A.

t_2 = time at B when signal received at B.

S = time required for transmission of signal.

The times obtained from the clocks at A and B after correcting the errors of the clocks and making necessary adjustments. Difference in longitude = $(t_1 + S) - t_2$ .....(1)

Again, t'_2 = time at B when the signal sent out in the reverse direction from B to A.

t'_1 = time at A when the signal received at A

S = time required for transmission.

Then, difference of longitude = $(t'_1 - t'_2 + S)$ .....(2)

By averaging (1) and (2),

$$\text{The difference of longitude} = \frac{1}{2}\{(t_1 + S - t_2) + (t'_1 - t'_2 - S)\}$$

$$= \frac{1}{2}\{(t_1 - t_2) + (t'_1 - t'_2)\}$$

The error due to the time required for transmission of signal being entirely eliminated and difference of longitude is very accurately determined. Now, if the longitude of one place is known, the other can be found out corrrectly. For each 4 seconds difference in time between two places, the difference of the longitude is 1 minute.

Question 16. Write short notes on :

(a) Electronic Distance Measuring Device.

(b) Trilateration.

Answer : SHORT NOTES ON E.D.M. AND TRILATERATION

(a) **Electronic Distance Measuring Device :** A major advance in surveying in recent year has been the development of Electronic Distance Measuring (EDM) Devices to determine the lengths based upon the time it takes electromagnetic energy to travel from one end of a line to the other and return. With modern EDM devices, distances are automatically displayed in digital form in feet or metres, some gives results internally reduced to horizontal. Several EDM instruments also measure angles to a few seconds of arc and can be attached to theodolites so that both angular and linear measurements can be ontained with the same instrument from one setting. The various instruments that are used under electronic methods are–geodimeter, based on the propagation of modulated light waves, and Tellurometer, dacca Navigator, Lambda position Fixing System, based on radio waves.

(b) **Trilateration :** Trilateration is based on the trigonimetrical proposition that if the three sides (laterals) of a triangle are known, the three angles can be computed.

Trilateration survey consists of establishing well-conditioned triangles on ground to form the strong geometrical figures as in triangulation survey and measuring the lengths of all lines directly by EDM, without measuring any horizontal angles in the network. The length of lines should be measured several times in order to maintain accuracy in the system.

Before the lengths of the lines in trilateration net can be used in any subsequent computations, their slope lengths determined by the instrument and corrected for atmospheric conditions must be reduced to the corresponding sea-level distances. The equivalent horizontal lengths of each line can be determined either from reciprocal vertical angles measured at the two ends of each line or the elevations of two ends of each line. The effects of atmospheric conditions, i.e. temperature, pressure and vapour pressure, on both light and microwaves are corrected by the meteorological data determined at the time of distance measurement. Some EDM instrument circuitry has been modified to maintain a constant wave length at any temperature and pressure and those display the corrected lengths directly.

Computing the horizontal lengths, the angles of all the triangles in the network are obtained using trigonimetric formula in the triangles. The obtained values of all the angles in the network should be subjected to the conditions of triangles, quadrilaterals or polygons. The error is then adjusted by approximate method or least square method. After adjusting all the angles both geometrically and trigonimetrically, the computations of the azimuths of all the lines in the network are made. As a check, the azimuth of the starting point must be the same when recalculated by different routes.

The lengths of the sides and azimuths are recomputed to be consistent with adjusted angles. The co-ordinates of all the control points in the trilateration system are then determined.

Trilateration may be combined with triangulation in order to strenghen a net or trilateration may be used as a check bases in a chain of figures to meet the criteria for strength of figures.

Question 17. Classify the elctronic distance measuring instruments. Explain the principle of operation of an electonic distance / linear measuring device.

Answer : CLASSIFICATION AND PRINCIPLE OF OPERATION OF EDM

Classification : The classification of electronic distance measuring instruments can be made by their range capabilities, although this is somewhat arbitrary. Such classification may be set out as follows –

1. Short-range EDM
2. Intermediate-range EDM
3. Long-range EDM.

A *short-range instrument* is one that is capable of measuring distances upto about 3 Km. The upper range of some EDM's is only 1000 m to 1500 m. The very short range instruments, however, incorporate features such as automatic reading, light weight, very low power requirements, and adaptability to angle measuring surveing instruments. For example : (a) H - P. 3800 distance meter of Hewlett Packard Co. (b) Wild D13 distance instrument which may be used with Wild T16 theodolite both of Wild Heebrugg Instruments, (c) Wild DI – 10 distomat, (d) Zeiss Reg Elta 14 electronic recording Tacheometer-Rangefinder, (e) Kern DM - 1000 EDM, (f) Kern MDM 500 minimeter, (g) AGA Model 76 Geodimeter are all classed as short range. These instruments all use light-emiting diodes to generate infrared light in the 900 to 930 -nm wave length region. The infrared light is referred to as the carrier. The instrument AGA-76 Geodimeter operates with visible laser light as the carrier.

An *intermediate-range instrument* is one that is capable of measuring over a range up to 12 km to 15 km. Different types of carriers are used in these instruments including tungsten light, mercury light and microwaves with frequencies of 10-billion Hz or 10 gigaherz (10 GHz). Laser light is used as carrier for the instruments having a range from 5 km to 12 km. Examples : AGA Model 6B/6BL Geodimeter, K & E Ranger III.

The *long-range instruments* in which light is used as the carrier can measure distances up to 60 Km with extremely high accuracy. The first geodimetres as well as new laser instruments are in this category. AGA Model 8 Geodimeter and K & E Rangemaster use laser light as the carrier and are capable of 60 km distances. The first microwave instrument Tellurometer M/RAI used for measuring distances with a range of 75 km to accuracies of 1 : 100,000 or better.

Measurement Principles of EDM using Microwaves : The microwave instruments, such as **Tellurometer M/RA-I,** gemerate Superhigh-Frequency (SHF) or extremely high-frequency (EHF) electromagnetic waves in the range of 3 to 35 GHz as the carrier waves. These in turn, are modulated to frequencies varying from 10 to 75 MHz, depending on the type of instrument. The length of the modulated wave is given by $\lambda = Vr/f$ in which λ is the wave length of modulation in metres, Vr is the velocity of the micro wave through the atmosphere, in metres per second and f is the modulating frequency in hertz. The value of Vr depends on prevailing temperature, atmospheric pressure, and the partial pressure of water vapour in the atmosphere. Two similar instruments are used in measurement with the microwave instrument which must also have an operator, serves like a reflector in the light-generating EDM's.

The operator at the master station selects a modulating frequency at which the microwaves are transmitted toward the remote station. He indicates to the remote operator by voice communication, which is built into the instrument, the transmitting frequency being used. The remote operator sets his instrument to correspond to that frequency. The signal is received by the remote instrument and is retransmitted back to the master with no delay. A phase meter at the master, the output of which is either displayed on an oscilloscope in the early microwave EDM (Tellurometer M/RA-1) or in digital form on the later models, gives the total amount by which outgoing and incoming signals are out of phase. This gives the fractional or decimal part of the wave length by which the double path from master to remote to master departs from an integral number of wavelengths. On the older instruments minor calculations are necessary to reduce the phase-meter readings to the desired distance, whereas the newer instruments provide automitic or semiautomatic reduction.

Question 18. What is the principle of trilateration surveying ? Describe any modern instrument by which you can conduct a trilateration survey on a huge mining property.

Answer : TRILATERATION SURVEY ON A MINING PROPERTY

Principle of Trilateration Survey : The principle of trilateration survey is based on the trigonometric proposition. The system consists in construction on the ground of a continuous chain of triangles and braced quadrilaterals in which all the sides and diagonals are measured directly using the electronic distance measuring devices without measuring horizontal angles in the network. The angles of all the triangles are obtained from the three sides (laterals) of respective

triangles by using trigonometric formula, and co-ordinates of all the control points are computed.

Wild D1-60 Distomat : The instrument measures from 20 m to about 150 km and uses a 3 cm carrier waves of frequencies 10324.3 MHz (with AFC) for the master and 10335 MHz (fixed) for remote and measuring frequencies of 149.848300 MHz for fine measurements and 14.984836 MHz, 14.977337 MHz, 14834932 MHz and 13.486347 MHz for coarse measurements.

The base of the D1 60 is the well known centring flange which fits into all tribrachs and therefore offers complete interchangeability and guaranteed forced centring with all Wild theodolites. The optical plummet in the tribrach enables the Distomat to be centred accurately and quickly.

The 3 cm carrier wave and the diameter of the antenna result in a beam of 6^0, thus searching for and finding the direction of the opposite station is extremely quick even when bad visibility prevents an approximate alignment being known. Standing and tilting axis permits it to be pointed like a theodolite telescope.

On pressing down a switch, D1 60 at the master station automatically tunes to the remote instrument in 2 to 3 seconds. Identical instruments are used at each station, their role being reversed by operating a switch.

For communication loudspeaker and magnetic microphone are provided. At the touch of the switch the measuring process is triggered off and the instrument automatically measures the phase shift of each of the five different modulation frequencies. The frequencies used give scales of 1 m, 10 m, 100, 1000 and 20,000 m. At the end of the measuring process, the distance is displayed immediately as an 8-figure number with the millimetres as the last figure. The read out is unambiguous up to 19999.99 m. For longer ranges distance must be known to the nearest 20 km, the multiple of 20 km contained in the distance being added to the displayed result. Because the D1 60 uses the extremely high modulated frequency of 150 MHz for the fine measurement, ground reflections have hardly any significance and therefore the accuracy is higher than that of other microwave equipment. The weight of the complete packed up equipment is only 13.5 kg and 12 volt battery locally available can be used.

Question 19. Name any two electronic distance measuring instruments and describe one briefly. What are the uses of EDM's in modern mine surveying ?

Answer : ELECTRONIC DISTANCE MEASURING INSTRUMENTS AND THEIR USES IN MINE SURVEYING

EDM Instruments : The electronic distance measuring devices are a new development in the field of surveying. The first such instrument *Geodimeter* was available to the general surveying and engineering profession in the early 1950s. The model employed a modulated light beam for determining distances. It was followed afterwards by the *Tellurometer* which employed modulated microwave. The advantage of microwave instruments is their operability in fog or moderate rain, day or night, as well as their generally longer range. The development and perfection of small light-emitting diodes in the mid 1960s as well as miniaturisation of electronics using solid-state components caused a revolution in the design of EDM's which are more portable, take less power and simpler to operate and read. The latest generation of EDM's employing highly coherent laser light has been brought the instruments to perfection in recent years.

Name of two EDM's are–

1. **Geodimeter (Model 8)** using laser powered by 12V battery. Maximum range 60 km in day or night with an accuracy ± 6 mm + 1 ppm.
2. **Tellurometer (Model MRA 4),** microwave system, distance upto 50 km can be determined uniquely with an accuracy ± 5mm ± 3 ppm, operated by 12V battery.

GEODIMETER : Geodimeter or Geodetic Distance Meter is a modern linear measuring equipment. The principle of Geodimeter is based on the transmission and reception of light waves.

A modulated light beam is transmitted by this instrument to a reflector, which is placed at the other extremity of the distance to be measured. The reflector transmits the light back to the Geodimeter where a phase comparison is made between the outgoing and ingoing light pulses. A measurement of number of light pulses and fractions that occur in this distance is obtained and the linear distance can then be obtained.

The beam sent out by the Geodimeter is modulated by means of a high frequency signal applied to a kerr cell, which acts as very rapid electronic shutter. A photo tube receives and converts the reflected light into electric signal. The phase relation between the transmitted and received light pulses is determined by an adjustable electronic delay line, inserted between the kerr cell and the photo tube. When the correct adjustment has been made, a null reading is obtained on a special instrument. The delay line readings are used for the calculation of the linear distance.

The operating time for a single observation with this instrument is only a few minutes and several readings can, therefore, be taken in a short period and with a high standard of accuracy. The latest Geodimeter (Model 8) uses a light from Helium Neon laser powered by 12V battery, thus converting it to a long range instrument. This improves a maximum range to 60 km with an accuracy ± 6 mm +1 ppm.

Uses of EDM in Modern Mine Surveying

The uses of electronic distance measuring instruments may be summarised as follows –

1. Establishment of control points by trilateration that is by the measurement of the lengths of sides of triangles only.
2. Measuring the base line and stiffening a triangulation network.
3. Rapid establishment of ground control points for photogrammetry.
4. Measurement of traverse lines precisely within a very short period.
5. Measuring the depth of shaft.

WORKED OUT EXAMPLES

Example 1. The following angles were measured by closing the horizon at a station. Make necessary station adjustment.

a = 104°36'42"
b = 63°57'26"
c = 85°42'41"
d = 105°43'03".

Answer : In closing the horizon the total angle should be 360°. But the angles (a + b + c + d) = 359°59'52". The deffciency of 8 seconds must be distributed proportionally among the four angles so that their sum is equal to 360°, thus :

a = 104°36'42" + 2" = 104°36'44"
b = 63°57'26" + 2" = 63°57'28"
c = 85°42'41" + 2" = 85°42'43"
d = 105°43'03" + 2" = 105°43'05"
360°00'00"

Example 2. The following angles were measured at a station. Make the necessary station adjustment.

a = 24°46'32"
b = 67°23'05"
c = 54°19'43"
d = (a + b + c) = 146°29'32"

Answer : The sum of the three angles,

a = 24°46'32"
b = 67°23'05"
c = 54°19'43"
a + b + c = 146°29'20"
and, d = 146°29'32"

Thus giving a difference ot 12" sec. All four angles are measured, however, and are equally liable to error so that the same adjustment must be made to each, namely 3 seconds. In the case of a, b and c the correction will be positive and in that of d, negative, so that :

a = 24°46'32" + 3" = 24°46'35"
b = 67°23'05" + 3" = 67°23'08"
c = 54°19'43" + 3" = 54°19'46"
146°29'29"
d = 146°29'32" – 3" 146°29'29"

Example 3. The following angles and check angles were measured at a station. Make the necessary adjustments.

a = 57°26'43"
b = 104°21'19"
c = 106°14'52"
d = 91°57'12"
e = (a + b) = 161°48'06"
f = (c + d) = 198°12'09"

Answer : This adjustment can be made statisfactorily only by the method of least squares, but there are two possible methods by equal shifts. The simple correction can be made by adjusting the check angles to equal 360° and then adjusting the individual angles to the corrected check angles.

e + f = 360°00'15"
Hence, differencnce = 15" or, 7.5" per angle.
Therefore, e = 161°48'06" – 7.5" = 161°47'58.5"
and f = 198°12'09" – 7.5" = 198°12'01.5"
360°00'00"

a + b = 161°48'02"
e (adjusted) = 161°47'58.5"
difference = 3.5" or; 1.75" per angle
Therefore, a = 57°26'43" – 1.75" = 57°26' 41.25"
b = 104°21'19" – 1.75" = 104°21' 17.25"
161°47' 58.5"

c + d = 198°12'04"
f (adjusted) = 198°12'01.5"
difference = 02.5" or; 1.25" per angle
Therefore, c = 106°14'52" – 1.25" = 106°14'50.75"
d = 91°57'12" – 1.25" = 91°57'10.75"
198°12'01.50"

The conditions have now been satisfied –

$$a + b + c + d = 360^\circ$$
$$e + f = 360^\circ$$
$$a + b = e$$
and $$c + d = f$$

Example 4. The following angles were measured at a triangulation station by closing the horizon. Make the necessary station adjustment.

a = 104°36'42" c = 85°42'41"
b = 63°57'26" d = 105°43'03"

Answer : Necessary station adjustment is tabulated below–

Angle	Value of Angle	Correction	Adjusted Value
a	104°36'42"	+ 02"	104°36'44"
b	63°57'26"	+ 02"	63°57'28"
c	85°42'41"	+ 02"	85°42'43"
d	105°43'03"	+ 02"	105°43'05"
	359°59'52"		Total = 360°00'00"
	360°00'00"		
	Correction = + 08"		

Question 5. The triangulation survey of a colliery is in form of a quadrilateral ABCD with AC and BD as the diagonals. The following mean value of included angles are observed :

1. Angle BAC = 63°35'43" 2. Angle ABD = 40°53'38"
3. Angle CBD = 34°29'53" 4. Angle BCA = 41°00'28"
5. Angle ACD = 45°32'18" 6. Angle CDB = 58°57'08"
7. Angle BDA = 37°48'48" 8. Angle DAC = 37°41'47"

State the various adjustments to be made to angles so that the figure of quadrilateral is geometrically correct and then make any two adjustments to the above angles.

Answer : Adjustment for angle condition in quadrilateral–

Let $\angle BAC = \alpha_1$, $\angle ABD = \beta_1$, $\angle CBD = \alpha_2$, $\angle BCA = \beta_2$, $\angle ACD = \alpha_3$, $\angle CBD = \beta_3$, $\angle ADB = \alpha_4$, $\angle CAD = \beta_4$, then :

(1) The sum of all internal angles = 360°. If the sum is more or less than 360°, the discrepancy should be equally distributed in all angles .

(2) $\alpha_1 + \beta_1 = \alpha_3 + \beta_3$ and $\alpha_2 + \beta_2 = \alpha_4 + \beta_4$

Any discrepancy should be equally distributed in angles forming the equation.

(3) $\alpha_1 + \beta_1 + \alpha_2 + \beta_2 = \alpha_2 + \beta_2 + \alpha_3 + \beta_3 = \alpha_3 + \beta_3 + \alpha_4 + \beta_4$

$= \alpha_4 + \beta_4 + \alpha_1 + \beta_1 = 180^\circ$.

These conditions are automatically satisfied if conditions (1) and (2) are satisfied.

Calculation of 360° and angle Condition Adjustment :

Angle	Value of Angle	Equation for Diff. Δs	Discrepancy	Adjustment Opposite Angle	360° Adjust-ment	Total Adjust-ment	Adjusted Value of Angles
α_1	63°35'43"	104°29'21"		+ 2"	+ 2"	+ 4"	63°35'47"
β_1	40°53'38"			+ 1	+ 2	+ 3	40°53'41"
			+ 5"				
α_3	45°32'18"	104°29'26"		− 1"	+ 2"	+ 1"	45°32'19"
β_3	58°57'08"			− 1"	+ 2"	+ 1"	58°57'09"
α_2	34°29'53"	75°30'21"		+ 3"	+ 2"	+ 5"	34°29'58"
β_2	41°00'28"			+ 4"	+ 2"	+ 6"	41°00'34"
			+ 14"				
α_4	37°48'48"	75°30'35"		− 3"	+ 2"	− 1"	37°48'47"
β_4	37°41'57"			− 4"	+ 2"	− 2"	37°41'45"
	359°59'43"			+ 1"	+ 16"	+ 17"	360°00'00"
	360°00'00"						
	17"						
	− 01"						
	16"						

Example 6. The triangulation of a colliery is in form of a quadrilateral ABCD with AC and BD as diagonals. The following mean values of the included angles are observed :

At Station A	1 = 44°35'47"	2 = 31°36'32"
At Station B	3 = 70°36'48"	4 = 43°35'42"
At Station C	5 = 34°11'25"	6 = 71°52'52"
At Station D	7 = 30°20'13"	8 = 33°11'08"

What are the various adjustments that should be made so that the figure of the quadrilateral is geometrically correct. Make any three adjustments to the above angles assuming that all the angles are of equal weights.

Answer : Calculation of 180°, 360° and Angle condition adjustments –

Angle	Value of Angle	Equation for Diff. Δs	Discrepancy	Adjustment Opposite Angles	360° Adjust-ment	Total Adjust-ment	Adjusted Value of Angles
1	44°35'47"	77°46'55"		+ 3"	− 3"	0	44°35'47"
8	33°11'08"			+ 3"	− 3"	0	33°11'08"
			+ 12				
5	34°11'25"	77°47'07"		− 3"	− 3"	− 6	34°11'19"
4	43°35'42"			− 3"	− 3"	− 6	43°35'36"
2	31°36'32"	102°13'20"		− 4"	− 3"	− 7	31°36'25"
3	70°36'48"			− 4"	− 4"	− 8	70°36'40"
			+ 15				
6	71°52'52"	102°13'05"		+4"	− 4"	0	71°52'52"
7	30°20'13"			+ 3"	− 3"	0	30°20'13"
	360°00'27"			+ 1"	− 26"	+ 27"	360°00'00"
	Diff + 27"						
	− 1"						
	Dis. − 26"						

Since opposite angle and 360° condition is satisfied, 180° Adjustment is automatically satisfied.

Example 7. A tape 20 m long of standard length at 84°F was used to measure a line, the mean temperature during measurement being 65°F. The measured distance was 882.10metres, the following being the slopes :

2°20' for 100 m
4°12' for 150 m
1°06' for 50 m
7°48' for 200 m
3°00' for 300 m
5°10' for 82.10 m

Find the true length of the line if the co-efficient of expansion is 65×10^{-6} per 1°F.

Answer : Correction for temperature for the whole length of the measured distance
$= L\,\alpha\,(T - To) = 812.10 \times 65 \times 10^{-6}\,(65 - 84) = 0.109$ m (subtractive).

Correction for slope $= \Sigma l\,(1 - \cos\theta)$

$= 100\,(1 - \cos 2°20') + 150\,(1 - \cos 4°12') + 50\,(1 - \cos 1°06') + 200\,(1 - \cos 7°48') + 300\,(1 - \cos 3°) + 82.10\,(1 - \cos 5°10')$

$= 0.0715 + 0.4028 + 0.0092 + 1.8504 + 0.4111 + 0.3336$

$= 3.078$ m (subtractive)

Total correction = 0.109 + 3.078 = 3.187 m (subtractive)

Hence **Corrected Length** = 882.10 – 3.187 = **878.913 metres.**

Example 8. Calculate the sag correction for a 30 m steel tape under a pull of 10 Kg in three equal spans of 10 m each. Weight of 1 cubic cm of steel = 7.86 gm. Area of cross-section of the tape = 0.08 Sq. cm.

Answer : Volume of tape per metre run = 0.08 x 100 = 8 cubic cm.

∴ Weight of tape per metre run = 8 x 7.86 gm.

∴ Total weight of the tape suspended between two supports (for a span of 10 m) = 8 x 7.86 x 10 = 628.8 gm = 0.6288 Kg.

Now, correction for sag = $\frac{nl_1 (wl_1)^2}{24P^2}$ meters (negative)

where n = number of bays; l_1 = length of bay suspended between the supports, in metres; w = weight of tape in kg per metre run; P = pull applied in kg

Given, n = 3; l_1 = 10 metres; wl_1 = 0.6288 kg and P = 10 kgs.

∴ **correction for sag** = $\frac{3 \times 10\ (0.6288)^2}{24 \times 10^2}$ 0.00494 m.

Example 9. A base line was measured with a steel tape which was exactly 30 m at 18°C and a pull fo 5 Kg, and the measurud length was 459.242 m. Temperature during measurement was 28°C and the pull applied was 10 Kg. The tape was uniformly supported during the measurement. Find the length of the line if the crosssectional area of tape was 0.02 cm², the co-efficient of expansion per 1°C = 0.0000035, the modulus of elasticity = 2.1 x 10^6 Kg per cm².

Answer : Correction for temperature per tape length

= Lα (T– T°)

= 30 x 0.0000035 (28 – 18) = 0.00105 m (+ve)

Sag correction per tape length = 0.00

Correction for pull per tape length

= $\frac{(Pm - Po)\ L}{AE} = \frac{(10 - 5) \times 30}{(0.02)\ (2.1 \times 10^6)}$ = 0.00357 m (+ve)

Total correction per tape length = + 0.00462 m

True length of the tape = (30 + 0.00462) = 30.00462 m

Hence **the true length of the base line**

= 459.292 x $\frac{30.00462}{30}$ = **459.342 metres.**

Example 10. A tape of 90 m nominal length was standardised on the flat and its true length was found to be 90.005 m at a temperature of 75° F. The tape was then used in catenary, in three equal spaces of 30 m each to measure a level line, the apparent length of which was found to be 809.30 m. The weight of the tape was 336 gm per 30 m length and the pull used both during

standardisation and field measurement was 7.2 kg. The altitude of teh base line above sea level was 540 m. Assuming that mean temperature during the field measurement was 60°F and that co-efficient of expansion = 0.0000062 per °F, calculate correct length of the line.

Answer : The following corrections are applied to get correct length of base.

(a) **Correction for Tension :** Since tension used is same both during standardisation and field measurement no correction is required for Tension.

(b) **Correction for sag :** It is given by the formula –

correction, S = $\frac{W^2L}{24P^2}$ When L = Length of tape in catenary.

W = Weight of tape in catenary in kg and

P = Tension = 7.2 kg.

∴ S = $\frac{0.346^2 \times 30}{24 \times 7.2^2}$ = 0.0027 m per 30 m length

To measure 809.30 m length 26 spans of 30 m length covers 780 m and the last span measures (809.30 – 780) = 29.30 m

Thus sag correction for 780 m = 0.0027 x 26 = 0.0702 m and for 29.30 m –

Weight of tape for 29.30 m length = $\frac{0.336}{30}$ x 29.30 = 0.32816 Kg.

Hence sag correction for 29.30 m = $\frac{(0.32816)^2 \times 29.30}{24 \times 7.2^2}$ = 0.002536 m

Total correction for sag = 0.0702 + 0.002536 = **0.072736 m.**

(c) **Correction for temperature** is given by

Lα (T – T_o) = 809.30 x 0.0000062 x (60 – 75) = 0.0752649 m (–ve)

(d) **Correction for standardisation** = 0.005 x $\frac{899.30}{90}$

= 0.0449611 metres (+ve)

Corrected length of base after allowing for temperatnre, sag and standardisation

= 809.30 – 0.072736 – 0.0752649 + 0.0449611 = 809.19734 m.

Now, correction for altitude is given by –

Correction = $\frac{Lh}{B}$ where L = Measured length of base in metres, h = Average height of base above M.S.L, and R = Mean radius of earth in metres.

= $\frac{809.19434 \times 540}{6371000}$ = 0.0685768 metres.

Hence **true length of the line as referred to M.S.L.** = 809.19734 – 0.0685868 = **809.12875 Metres.**

Example 11. A steel tape 30 m long, standardised at 12°C with a pull of 12 kg, was used for measuring a base line. Find the correction per tape length, if the temp. at the time of measurement was 22°C and the pull applied was 17 kg. Assume weight of a cubic cm of steel = 8 gms; weight of tape = 600 gms; Co-efficient of thermal expansion of tape per 1°C = 12×10^{-6}; and E = 21×10^5 kg per sq. cm.

Answer : (a) Correction for temperature = α (Tm – To) L

Given, $\alpha = 12 \times 10^{-6}$; Tm = 22°C; To = 12°C; L = 30 m.

$\therefore$ Correction = $12 \times 10^{-6}(22^0 - 12^0) \times 30 = 0.0036$ (+ve)

(b) Correction for pull = $\frac{(P - Po)\, L}{AE}$

Given, P = 17 kg; Po = 12 kg; L = 30 m; E = 21×10^5 kg per sq. cm.

Weight of tape = A x 30 x 100 x 8 gms = 600 gms (given)

$$A = \frac{600}{30 \times 100 \times 8} = 0.025 \text{ sq. cm.}$$

$$\therefore \text{Correction} = \frac{(17 - 12) \times 30}{0.025 \times 21 \times 10^5} = 0.0029 \text{ m (+ve)}$$

(c) Correction for sag = $\frac{LW^2}{24\, P^2}$

Given, W = Weight of tape in kg = 0.6 kg.

P = Applied pull = 17 kg.

$$\therefore \text{Correction} = \frac{30 \times (0.6)^2}{24 \times 17^2} = 0.0015 \text{ m (–ve)}$$

Total correction to be applied per tape length

0.0036 + 0.0029 – 0.0015 = 0.005 m (+ve) Ans.

Example 12. A base line is measured in six number of drafts and the details of measurements are given below :

Drafts	Length in metres	Mean temp. in Degrees F	Staff readings in metres	Tension
0 – 1	29.672	71°	1.192 at 0	Stadard
1 – 2	29.441	70.5°	1.203 at 1	at both
2 – 3	28.507	70°	1.189 at 2	ends of
3 – 4	29.755	69.5°	1.185 at 3	each
4 – 5	29.000	71.5°	2.046 at 4	draft
5 – 6	29.250	70°	2.098 at 5	
			2.140 at 6	

Calculate the length of the base assuming co-efficient of expansion of steel to be 0.0000065 per degree F and the standardisation temp of the tape 66°F.

Answer : (a) Correction for temp. = α (Tm – T_0) L

where α = Co-efficient of thermal expansion.

Tm = mean temp. during measurement.

T_0 = temp. at which the tape is standardized.

L = measured length.

This correction is plus or minus according as Tm is greater or less than T_0. Since the mean temp. at the time of measurements are all above that at which the tape was standard. Corrections for temp for each draft are positive.

(b) Correction for slope = $\frac{h^2}{2L}$(–ve)

where h = difference of elevation between two end of the draft.

L = Length of the draft measured.

The results tabulated as under :

Draft	Measured length (m)	Corrections required	Total correction (m)	Corrected Length of Draft (m)
0 – 1	29.672	(a) Correction for temp = 0.0000065 (71° – 66°) 29.672 = 0.00096434 (+ve) (b) Correction for slope = $\frac{(1.203 - 1.192)^2}{2 \times 29.672}$ = 0.0000020338 m (–ve)	+ 0.00096	29.67296
1 – 2	29.441	(a) Corr. for temp = 0.0000065 (70.5 – 66) x 29.441 = 0.0008631 m (+ve) (b) Correction for slope = $\frac{(1.203 - 1.189)^2}{2 \times 29.441}$ = 0.0000033287 m (–ve)	+ 0.00086	29.44186
2 – 3	28.507	(a) Corr. for temp = 0.0000065 x (70° – 66°) 28.507 = 0.00074115m (+ve) (b) Correction for slope = $\frac{(1.189 - 1.185)^2}{2 \times 28.507}$ = 0.00000028 m (–ve)	+ 0.00074	28.50774
3 – 4	29.755	(a) Correction for temp = 0.0000065 x (69.5° – 66°) 29.755 = 0.00067695 m (+ve) (b) Correction for slope = $\frac{(2.046 - 1.185)^2}{2 \times 29.755}$ = 0.012456 m (–ve)	– 0.01178	29.74322

(Contd.)

Draft	Measured length (m)	Corrections required	Total correction (m)	Corrected Length of Draft (m)
4 – 5	29.000	(a) Correction for temp = 0.0000065 x (71.5° – 66°) 29 = 0.0010367 m (+ve) (b) Corrcetion for slope = $\frac{(2.098 - 2.046)^2}{2 \times 29}$ = 0.0000466 m (–ve)	+ 0.00099	29.09009
5 – 6	26.250	(a) Correction for temp = 0.0000065 x (70° – 66°) 26.250 = 0.000682 m (+ve) (b) Correction for slope = $\frac{(2.140 - 2.098)^2}{2 \times 26.507}$ = 0.0000336 m (–ve)	+ 0.00065	26.25065
1 to 6	172.625		– 0.00758	172.61742

Actual length of the base = 172.61742 m

Example 13. It was impossible to observe the length and bearing of the line PQ directly. The following observations were taken from two stations A and B –

Line	Length in metres	Bearing
AP	126.00	S 65°36'W
AB	314.00	N 24°12'E
BQ	115.50	N 76°48'W

Compute the length and bearing of PQ and the angles APQ and BOP.

Answer : Given the bearing of ling AP = S 65°36'W

or, the bearing of line PA = N 65°36'E

Line	Stn	Q. B. θ	Distance (m)	Log Dist. + Log Cos θ	Log Dist. + Log Sin θ	Partial Co-ordinates Lat	Partial Co-ordinates Dep
PA		N 65°36'E	126.00	2.10037	2.10037		
				1.61606	1.95937		
	A			1.71643	2.05974	+ 52.050	+ 114.746
AB		N 24°12'E	314.00	2.49693	2.49963		
				1.96035	1.61270		
	B			2.45698	2.10963	+ 286.40	+ 128.715
BQ		N 76°48'W	115.50	2.06255	2.06255		
				1.35860	1.98837		
	Q			1.42115	2.05092	+ 26.374	– 112.448
			Total Co-ordinates of Q			+364.824	+ 131.023

It is seen from the co-ordinates that Q is 364.824 metres North and 131.02 metre East.

Tangent of bearing PQ = $\frac{\text{Total dep.}}{\text{Total lat}} = \frac{131.023}{364.824} = 0.35914$

= tan 19°45'

∴ **Bearing of line PQ = N 19°45'E**

Length of line PQ = Dep. x Cosecθ

= 131.013 x Cosec 19°45'

Bearing of line PA = N 65°36'E

and, bearing of PQ = N 19°45'E

∴ **Angle APQ** = 65°36' – 19°45' = 45°51'

Bearing of line BQ = N 76°48'W

or, bearing of QB = S 76°48'E

and Bearing of line PQ = N 19°45'E

or, bearing of QP = S 19°45'W

∴ **Angle BPQ** = 76°48' + 19°45' = 96°33'

Example 14. The boundaries of mineral leasehold have been defined by stating the National Grid Co-ordinates or four points A, B,C and D, the area being bounded by straight line joining these points. For convenience the full grid co-ordinates have been reduced to an origin X and the co-ordinates in metres in relation to X are stated below :

Station	Easting	Northing
A	0	1250
B	910	3850
C	6170	4305
D	4880	0

Calculate from the co-ordinates, without plotting or scaling, the area of the leasehold in square metres. If the position of the leasehold in relation to the National Grid is such that the scale correction required is that 1000 metres on the grid would measure 1000.25 metres linear distance on the ground, find the corrected area in this case.

Answer : The simplest method of obtaining the area of a closed polygon from the co-ordinates of the corners is as follows :

Multiply the sum of adjacent total latitudes by the difference of adjacent total departures and divide the algebraic sum of the products by two.

Station	Total Lat.	Total Dep.	Sum of Lats.	Diff. of Deps.	Double Areas +	Double Areas −
A	1250	0	5100	910	4641000	
B	3850	910	8155	5260	42895300	
C	4305	6170	4305	− 1290		5553450
D	0	4880	1250	− 4880		6100000
					47536300	11653450
					11653450	
					2 | 35882850	
					17941425	

∴ **The area of leasehold = 17941425 sqare metes.**

Scale Correction :

Since the Scale Correction for the linear measurement is $\frac{1000.25}{1000}$ the correction for areas will be square of this.

Therefore, **corrected area** $= 17941425 \times (1000.25 + 1000)^2$

= 17950397 sq. metres.

Example 15. In a Snallious Rhombus ABCD, the following observations were taken :

value of apical angle ABC = 20°20'40"

value of apical angle ADC = 28°40'00"

value of angle BAC = 72°19'20"

value of angle BCA = 87°20'00"

value of angle DAC = 74°20'00"

value of angle DCA = 77°00'00"

Measured length of short base AC = 158.25 metres.

The bearing of the base is N 25°18'40"E. Calculate the length and bearing of the extended base BD.

Answer : Roughly plot the Rhombus ABCD.

Bearings : AC = N 25°18'40"E

AB = (360° + 25°18'40") − 72°19'20" = 312°59'20"

AD = 25°18'40" + 74°20'00" = 99°38'40"

In triangle ACB, $\frac{AB}{\sin C} = \frac{AC}{\sin B}$

$$\therefore AB = \frac{AC \times \sin C}{\sin D} = \frac{158.25 \times \sin 87°20'}{\sin 20°20'40"} = 454.6891 \text{ m}$$

$$\therefore AD = \frac{AC \times \sin C}{\sin D} = \frac{158.25 \times \sin 77°}{\sin 28°40'} = 321.4297 \text{ m}$$

From triangle ABD, we have, by cosine rule –

$$BD = \sqrt{AD^2 + AB^2 - 2 \times AD \times AB \times \cos (72°19'20" + 74°24")} = 744.4744 \text{ m}$$

In triangle ABD, $\frac{AD}{\sin B} = \frac{BD}{\sin A}$

$$\therefore \sin B = \frac{AD \times \sin A}{BD} = \frac{321.4297 \times \sin 146° 39'20"}{744.4744}$$

$\angle ABD = 13°43'42"$

Azimuth Bearing of AB = 312°59'20"

BA = (312°59'20" − 180°) = 132°59'20"

Bearing of BD = 132°59'20" − Angle ABD

= 132°59'20" − 13°43'42" = 119°15'38" = S 60°44'22"E.

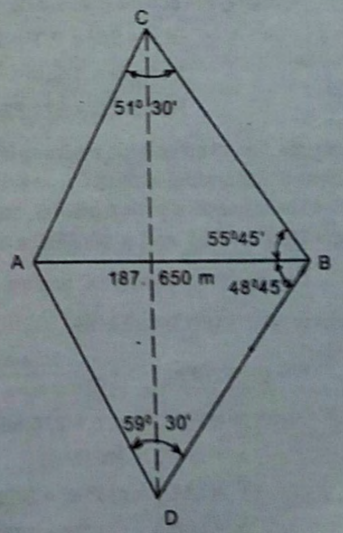
∴ **The length and bearing of extended base BD** are 744.4744 metres and S 60°44'22"E.

Example 16. The following observations were made for the extension of base line AB which has been accurately measured to be 187.650 metres in length :

Angle ABC = 55°45'; angle ACB = 51°30'; angle ABD = 48°45'; and angle ADB = 59°30'.

Calculate the extended base line CD.

Answer : EXTENSION OF BASE LINE



In triangle ABC,

Angle CAB = 180° − (51°30' + 55°45') = 72°45'

$$AC = \frac{187.650 \sin 55°45'}{\sin 51°30'} = 198.20 \text{ m}$$

In triangle ADB

Angle BAD = 180° − (48°45' + 59°30')

= 71°45'

$$AD = \frac{187.650 \sin 48°45'}{\sin 59°30'}$$

= 163.74 m

In triangle ADC, by applying cosine rule,

We get –

$CD^2 = AC^2 + AD^2 - 2\ AC \times AD \times \cos CAD$

$= (198.2)^2 + (163.74)^2 - 2 \times 198.2 \times 163.74 \times \cos (72°45' + 71°45')$

= 118935

$CD = \sqrt{118935} = 344.87$ m

∴ **length of extended base line CD = 344.87 metres.**

Example 17. Find the length of the side CD from given base and base angles :

Base AB = 584.5 metres :

Angle CAB = 67°30'

Angle DAB = 32°25'

Angle ABC = 44°18'

Angle ABD = 72.41'

Answer : In triangle ABC,

Angle ACB = $180^0 - (67°30' + 44°18') = 68°12'$

Applying the sine rule, $\frac{AB}{\text{Sine } 68°12'} = \frac{BC}{\text{Sin } 67°30'}$

$BC = \frac{584.5 \times \text{Sin } 67°30'}{\text{Sin } 68°12'} = 581.600$ metres.

In triangle ABD,

Angle ADB = $180^0 - (32°25' + 72°41) = 74°54'$

Applying the sine rule, AB / Sin 74°54' = BC / Sine 32°25'

$BD = \frac{584.5 \times \text{Sin } 32°25'}{\text{Sin } 74°54'} = 324.540$ metres.

In triangle CBD,

Angle CBD = 72°41' + 44°18' = 116°59'

Applying the cosine rule, we have :

$DC^2 = BC^2 + BD^2 - 2BC.BD.$ cosine angle CBD

Therefore, $DC. = \sqrt{581.6^2 + 324.54^2 - 2 \times 581.6 \times 324.54 \times \cos 116°59'}$

Hence DC = 521.82 **metres.**

Example 18. In measuring a survey line running approximately due East, an obstruction being at point B and ending at point C was encountered. The distanace AB and CD were 265.20 m and 275.40 m respectively. At a point E, the following angles were measured : angle BEA = 23°18', angle CEB = 37°24' and angle DBC = 31°18'.

Caulate the distance BC and the whole length of the line.

Answer : BROKEN BASE LINE

We know, $(a + x)(b + x) = \frac{ab \sin\alpha\, \beta \sin(\delta - \alpha)}{\sin\alpha \sin(\delta - \beta)}$

Given, a = 265.20 m; b = 275.40 m.

$\alpha = \angle AEB = 23°18'$

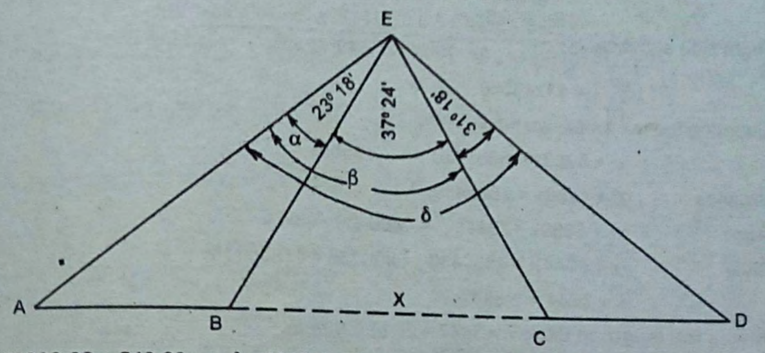
$\beta = \angle AEC = (23°18' + 37°24') = 60°42'$

$\delta = \angle AED = (23°18' + 37°24' + 31°18') = 92°00'.$

$\therefore (\delta - \alpha) = 68°42'$ and $(\delta - \beta) = 31°18'$

Putting respective values in the equation,

$$(265.20 + x)(275.40 + x) = \frac{265.2 \times 275.40 \times \text{Sin } 60°42' \text{ Sin } 68°42'}{\text{Sin } 23°18' \text{ Sin } 31°18'}$$



$73036.08 + 540.60x + x^2 = 288776.91$

$x^2 + 540.60x = 215740.83$

$x^2 + 540.60x + (270.30)^2 = 215740.83 + (270.30)^2$

$(x + 270.30)^2 = 288802.92 = (537.40)^2$

$x + 270.30 = 537.40$

$x = 537.40 - 270.30 = 267.10$ metres.

Therefore **the distance BC = 267.10 metres.**

Total length of Base line = 265.20 + 267.10 + 275.40 = **807.70 metres.**

Example 19. A straight line ABCD forming part of a triangulation survey can not be measured from end to end owing to an obstruction, The parts AB and CD as measured, and the angles at point O subtending the measured portions of the line are as follows :

AB = 1848 m; CD = 3440 m; angle BOA = 35°59'12"; angle COB = 21°05'05"; angle DOC = 49°39'59".

Calculate the length of the line BC.

Answer : It is desired to find the length of BC.

AB = a = 1848 m

CD = b = 3440 m

BC = x metres.

We have, $(a + x)(b + x) = \frac{ab \text{ Sin}\beta. \text{ Sin } (\delta - \alpha)}{\text{Sine } \alpha. \text{ Sin } (\delta - \beta)}$

Given, $\alpha = \angle AOB = 32°59'12"$

$\beta = \angle AOC = BOA + COB = 57°04'17"$

$\delta = \angle AOD = AOC + DOC = 106°44'16"$

$(\delta - \alpha) = 106°44'16" - 35°59'12" = 70°45'04"$

$(\delta - \beta) = 106°44'16" - 57°04'17" = 49°39'59"$

Left side equation = $x^2 + x(a + b) + ab$

$$x^2 + 5288x + 6357120$$

Right side equation = $\dfrac{1848 \times 3440 \times \text{Sin } 57°04'17" \times \text{Sin } 70°45'04"}{\text{Sin } 35°56'12" \times 49°39'59"}$

$$= 11246550$$

Equating the two sides, we have

$$x^2 + 5288x + 6357120 = 11246550$$

Whence $x^2 + 5288x = 4889430$

And $x^2 + 5288x + 2644^2 = 4889430 + 2644^2$

Thus $(x + 2644)^2 = 4889430 + 6990736 = 11880116$

$$x + 2644 = \pm 3447$$

Hence **the length of BC** = x = 3447 – 2644 = **803 m.**

Example 20. The horizontal distance between two triangulation stations A and B is 608.339 M. Station 'B' is due East of A. The under noted angles were measured at A and B to a station C which lies to the south of A and B.

Angle BAC = 93°16'32"

Angle CBA = 45°29'08"

A satellite station D is established to the east of C at a distance of 2.935 M and

Angle CDA = 46°23'30"

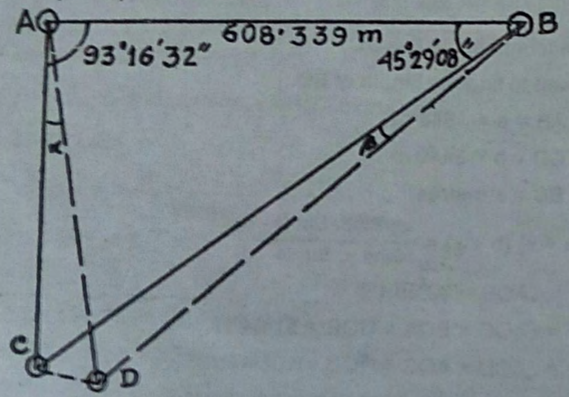
Angle ADB = 41°14'20"

Determine angle ACB and the error of closure of the triangle ABC.

Answer : ERROR OF CLOSURE OF TRIANGLE

In triangle ABC, ∠A = 93°16'32" and ∠B = 45°29'08"

Angle C = 180° – (A + B) = 41°14'20"



$$AC = \frac{AB \times \text{Sin } 45°29'08"}{\text{Sin } 41°24'20"} = 658.0561 \text{ m}$$

Let, ∠CAD = α and ∠CBD = β

$$\text{Sin } \alpha = \frac{CD \text{ Sin } CDA}{BC} = \frac{1935 \times \text{Sin } 46°23'30"}{658.0561} = \text{Sin } 11°10'20"$$

$$\text{Sin } \beta = \frac{CD \text{ Sin } CBD}{BC} = \frac{2.935 \times \text{Sin } (46°23'30" + 41°14'20")}{921.337} = \text{Sin } 11'00".54$$

Corrected angle C = 41°14'20" – α + β = 41°14'20" – 11'10".20 + 11'00".54

= 41°14'10".34

Now angle A = 93°16'32".00

B = 45°29'08".00

C = 41°14'10".34

Sum = 179°59'50".34

– 180°00'00".00

Difference = **– 00°00'09".66 = Error of closure of △ ABC**

Example 21. (a) What is a satellite station in Triangulation survey ? Discuss its use.

(b) From an eccentric station E, 13.8m from station A the angles measured to three trigonometrical stations A, B and C are as follows, the stations C and E being on opposite sides of the line AB :

∠BEC = 68°26'36" ; ∠CEA = 32°45'48".

The length of AC and AB are 5588.4m and 4371.0m respectively, calculate the angle BAC.

Answer : SATELLITE STATION

(a) **Satellite Station :** A satellite station is a subsidiary station near to the main station of triangulation survey from which observations are taken to the other triangulation stations with the same precision as would have been used in the measurement of angles at the true station. Satellite station is also known as eccentric station or false station.

In order to secure well-conditioned triangles or better visibility, objects such as church spires, flag poles, towers, etc. are sometimes selected as the triangulation stations. When the observations are to be taken from such a station, it is impossible to set up an instrument over it. For the continuation of triangulation the theodolite may have to be placed over the point occupied by the axis of the signal or the rays from the station are obstructed or a scaffold may have to be erected for the purpose. In all such cases, the necessary observations may be taken from the satellite station, and these observations therefrom can be adjusted to agree with these which would have been made from the main triangulation station. With best laid out triangulation it is never used and it can be easily avoided with little care.

(b) The positions of all the stations are roughly plotted.

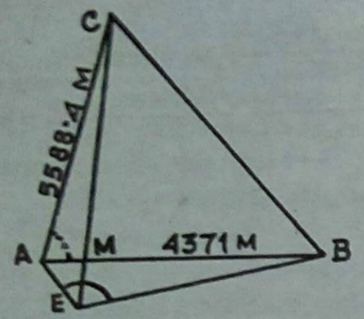
From the figure –

In triangle ABE :

∠AEB = 32°45'48" + 68°26'36" = 101°12'24"

$$\angle ABE = \text{Sin}^{-1} \frac{13.8 \times \text{Sin } 101°12'24"}{4371} = 00°10'39"$$

$\angle BAE = 180^\circ - (101^\circ12'24'' + 00^\circ10'39'') = 78^\circ36'57''$



$$\therefore BE = 4371 \times \frac{\text{Sin } 78^\circ36'57''}{\text{Sin } 101^\circ12'24''} = 4368.30\text{m}$$

In triangle MBE :

$\angle BME = 180^\circ - (68^\circ26'36'' + 00^\circ10'39'') = 111^\circ22'45''$

$$\therefore BM = \frac{4368.30 \times \text{Sin } 68^\circ26'36''}{\text{Sin } 111^\circ22'45''} = 4362.98 \text{ m}$$

$\therefore AM = 4371 - 4362.98 = 8.02$ m

$\angle AMC = \angle BME = 111^\circ22'45''$

$$\angle ACM = \text{Sin}^{-1} \frac{8.02 \times \text{Sin } 111^\circ22'45''}{5588.40} = 00^\circ04'36''$$

Hence $\angle BAC = 180^\circ - 00^\circ04'36'' = 68^\circ32'39''$

PROBLEMS

Problem 1. A steel tape 20 m long standardised at 55°F with a pull of 10 kg was used for measuring a base line. Find the correction per tape length, if the temperature at the time of measurement was 80°F and the pull exerted 16 kg. Weight of 1 cubic cm of steel = 7.86 gm. Weight of the tape = 0.8 kg. E = 2.109 x 10^6 kg per sq. cm. Co-efficient of expansion of tape per 1°F = 6.2 x 10^{-6} **[Ans. Total Correction = 0.00213 m]**

Problem 2. A nominal distance of 30 m was set out with a 30 m steel tape from a mark on the top of one peg to a mark on the top of another, the tape being in catenary under a pull of 10 kg and at a mean temperature of 70°F. The top of the higher peg was 460 m above mean sea level. Calculate the exact horizontal distance between the marks on the two pegs and reduce it to mean sea level. If the tape was standardised at a temperature of 60°F, in catenary under a pull of (a) 8 kg. (b) 12 kg. and (c) 10 kg. Take radius of earth = 6370 km : Density of tape = 7.86 gm/cm^3; Section of tape = 0.08 sq. cm ; co-efficient of expansion = 6 x 10^{-6} per 1°F and Young's modulus = 2 x 10^6 kg per cm^2. **[Ans. (a) + 0.02608 m ; (b) – 0.0132 m ; + 0.0008 m]**

Problem 3. Find the sag correction for 30 m steel tape under a pull of 8 kg in three equal sp

of 10 m each. Weight of one cubic cm of steel = 7.86 gm/cm^3. Area of cross-section of the tape = 0.10 sq. cm. **[Ans. 0.01206 m]**

Problem 4. A steel tape is 30 m long at a temperature of 65°F when lying horizontally on the ground . Its sectional area is 0.082 sq cm. its weight 2 kg and co-efficient of expansion 65 x 10^7 per 1°F. The tape is stretched over three equal spans. Calculate the actual length between the end graduations under the following conditions : temp 85°F, pull 18 kg. **[Ans. 30.005 m]**

Problem 5. A 30 m steel tape was standardised on the flat and was found to be exactly 30 m under no pull at 66° F. It was used in catenary to measure a base of 5 bays. The temperature during the measurement was 92°F and pull exerted during the measurement was 10 kg. The area of cross-section of the tape was 0.03 sq. cm. The specific weight of steel is 7.86 gm/cm^3.

α = 0.0000063 per 1°F and E = 2.109 x 10^6 kg/cm^2. Find the true length of the line. **[Ans. 30.005 m]**

Problem 6. A straight line, AD, running approximately due East, is obstructed between points B and C which are aligned in it. A station P is set up and the following angles are measured : Angle DPC = 45°20', CPB = 33°25', BPA = 29°05'.

The length of AB and CD are 250 m and 315 m respectively. Calculate the length of BC and the whole length of the line AD. **[Ans. 163.85 m; 728.85 m]**

Problem 7. A steel tape is 30 m long at temp of 15°C when lying horizontally on the ground. Its sectional area is 0.08 sq cm, its weight 1.8 kg and the co-efficient of expansion is 117 x 10^{-7} per 1°C. The tape is stretched over three supports which are at the same level and at equal intervals. Calculate the actual length between the end graduations under following conditions : Temperature = 25°C and Pull = 18 kg. **[Ans. 30.00361 m]**

Problem 8. A base line, AD, running approximately due East, is obstructed between points B and C which are aligned in it. A station P is set up and the following angles are measured :

Angle DPC = 49°39', Angle CPB = 21°05', Angle BPA = 35°59'.

The length of AB and CD are 184.80 m and 344 m respectively. Calculate the length of BC and the whole length of the line AD. **[Ans. BC = 80.30 m; AD = 609.10 m]**

Problem 9. A portion of a straight base line between B and C can not be directly measured due to some intervening obstruction. To determine its length two auxiliary points A and D are taken on the base on either side of BC and a theodolite is set up at a point P on the right side of the base line. The angles APB, BPC and CPD are measured and found to be 20°50'25", 140°15'20" and 28°32'10" respectively. Compute the length of BC, if the corrected lengths of AB and CD are 411.52 m and 549.55 m respectively. **[Ans. 563.86 m]**

Problem 10. A survey line BC, bearing approximately due East, can not be measured directly, and its length is to be determined by a combination of linear and angular measurements. For this purpose a station A was established 500 metres to the West of B, and a station D was established 500 metres to the East of C. Stations A,B,C and D being in the same straight line. From the station E, sited to the south of the line, the undernoted horizontal angles were measured :

Angles AEB = 32°41'20", BEC = 43°20'00", CED = 41°03'10".

Compute the length of the line BC. **[Ans. 325.0304 m]**

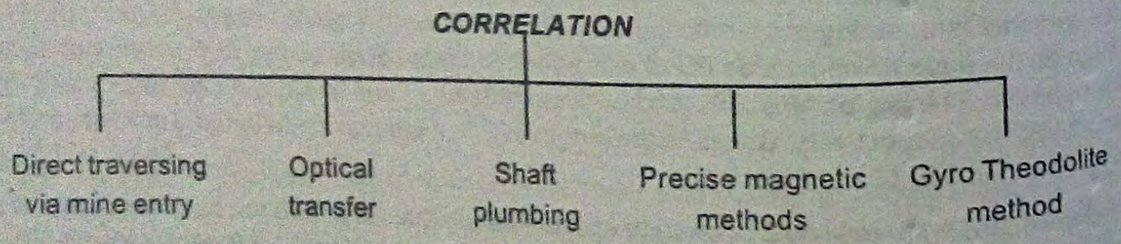
CHAPTER IV

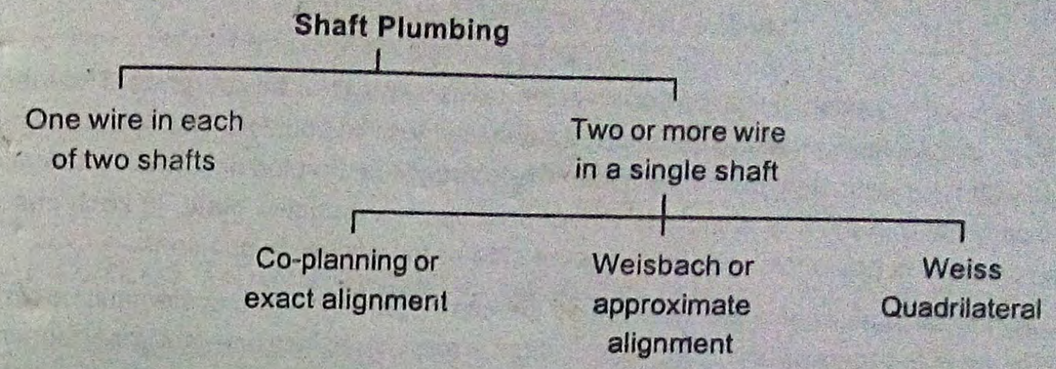
CORRELATION

CORRELATION OF SURFACE AND UNDERGROUND SURVEYS

The purpose of correlation Surveys is to determine with a high degree of exactitude the relative positions of the underground workings and the surface features so that the underground roadways and faces may be laid down accurately on the working plan and the statutory restrictions imposed may be strictly followed in mine workings. A high degree of accuracy is required in correlation survey otherwise safety and stability of the mine workings and of important surface features are very much jeopardized because of the disturbances and strata movement caused by mining. Mine correlation in the context of multilevel metalliferous mining, and horizontal coal mining is important from the point of view of accurately connecting two levels through such cross-measure drifts, or staple shafts or raises and winzes as are driven from both the levels. In nationalised coal industry, when a number of smaller units which have been amalgamated into bigger units will require interconnections, and connections through level galleries or inclined cross-measure drifts and roadways to centrally located such vital mine openings as will be used for ventilation and for winding and unless these connections are correctly established, advantages of concentration of mine workings would not be forthcoming.

Mine correlation involves determination of the bearing or azimuth of a line below ground with respect to a true north and also of the co-ordinates of one of the end stations of the line called **Reference Base**. The degree of accuracy required of such determination depends upon the extent of the mine workings, and also upon the purpose of correlation. The method of correlation depends upon the type of openings (inclines, adits, drifts, or vertical shafts), their number, and availability of trained and skilled surveyors. The **different methods of correlation are** classified as under :





On the surface the work involves the establishment of one or more stations either on the pit headgear or near the shaft or adit used for correlation. These stations must be accurately located by incorporating them in the triangulation network. If they can not form part of the main triangulation scheme a subsidiary triangle or quadrilateral from it might be formed. Long traverse from a trigonometrical station should be avoided.

DIRECT TRAVERSING THROUGH ADITS OR INCLINED DRIFTS

Where entry to a mine is gained by means of a level or drift, correlation may be carried out by making a precise theodolite traverse direct into the workings starting from some one line of known bearing in the surface triangulation and making a return traverse to the initial reference point so as to close the survey polygonally. In the course of the traverse, one or more underground base are established from which future underground surveys may be carried out. When the surveys are plotted on a plan, the underground workings will be shown in their true position relative to the surface features.

The traverse must be carried out with the requisite degree of precision, particular care being taken in (a) centring the theodolite at the various stations, (b) levelling the instrument, and (c) measuring the length each draft. The angle between each pair of adjacent lines should be measured at least three times on each face of the instrument, or by the method of repetition. The length of each draft should be measured several times, a spring balance being used to ensure correct tension and proper corrections made for sag unless the steel tape rests on the floor or is supported at short intervals. Special care is needed in measuring the vertical angles so that the slope length may be reduced to their correct horizontal equivalents.

Non-vertical shaft in metal mines, with inclination up to 85^0 from the horizontal, causes a most difficult problem in correlation. In mines where non-magnetic minerals are being worked the actual orientation of the underground base-line could be made by precise magnetic method. It would still be necessary to fix the co-ordinate position of some underground point, and as this can not be done by plumbing, a traverse must be made down the shaft. The sight will be exceptionally steep and special precautions must be taken in setting up the instrument and targets and in measuring the angles and traverse lines.

In such cases it will be impossible to use ordinary tripods and scaffolds must be erected in the shaft, one on which the observer can stand and another to carry a special tripod head which has spiked feet to grip the scaffold and a centering motion. Another form of support which can be

tightened against the sides of the road or shaft may also be used for this purpose. The theodolite itself can be moved laterally along the bar and set in any desired position. The upward sights can be made with the diagonal eyepiece, but when the telescope is directed downwards its inclination may be so great that the line of sight is intercepted by the theodolite plate. In such cases, the auxiliary telescope has to be used in either the side or top eccentric positions.

For normal work, where the inclinations are comparatively slight, systematic observation on both faces of the theodolite will cancel out slight errors due to instrumental imperfections and residual maladjustment. With steep sighting, however, errors arise which can neither be eliminated nor compensated by these methods.

In spite of appropriate adjustment having been made, the main error arises from failure of the transit axis to remain horizontal while observations are being made at a station. The cause may be the use of the relatively insensitive plate-bubbles for the final levelling, eccentricity between the inner and outer instrument centres, or movement of the instrument arising from instability of the set up. In each of these causes, dislevelment of the transit axis arises because the vertical axis is not truly vertical, and it follows, therefore, that the effect of inclination of the transit axis will be in the same direction irrespective of the face of the instrument and errors will be present in the measurement of the horizontal angles which can not be eliminated by normal systematic observations.

With inclination up to 45^0 the error in direction is less than the inclination of the transit axis, but as the inclination increases above this amount the error increases rapidly. With a tilt of 3 second in the transit axis, for example, at an inclination of 85^0, the error in the direction would be approximately 34 seconds. To eliminate this error the inclination of the transit axis should be accurately measured at each pointing and necessary correction to vernier reading calculated and applied.

Because of the short horizontal projections of steeply inclined sights, centring errors must be reduced to a minimum. It will normally be possible to plumb the instrument stations satisfactorily, the measuring heads should be set up at convenient points between the instrument setting, and the measurements made between them. If the inclinations are very steep and the correct tension is applied, the tape can hardly be considered to be hanging in catenary, and the measurements should need only the usual corrections for this class of work.

Permissible Limits of Error : The following standards have been laid down–

1. **The total angular errors of the closed traverse**–i.e. the difference between the sum of the observed interior angles of the enclosed figure and $(2n - 4) \times 90^0$, where 'n' is the number of observing stations– must not exceed $(20 + x)\sqrt{n}$ seconds, in which expression the value of 'x' is 10 second or the smallest reading in seconds of subdivisions of the circle of the instrument employed (whichever is the greater) and 'n' is the number of observing stations.

2. **The error of closure** of such a traverse (calculated by co-ordinates after distribution of the angular error) must not exceed 1 / 2500th of the sum of the horizontal length of the drafts of the traverse.

3. **The correlation** shall be deemed to be within the required limit of error when (after distribution of the permissible angular error in the traverse) the values of the azimuth of any underground reference line relative to the surface reference base line, as determined by any two or more such independent surveys, agree within a limit of error not exceeding **one minute of arc.**

DIRECT TRANSFERENCE OF AZIMUTH DOWN THE SHAFT

The method involves sighting either up and down the mine shaft with a transit instrument of some special form of telescope. Although not in current use, such method has been much used in the past in shallow shafts. The procedure is as follows –

(a) Set out a base line in the pit bottom connected to the underground survey and line in positions of two points A and B across the shift bottom.

(b) Plumb the theodolite over B, align the side with the base line and transit up to two targets F and G placed across the shaft-top on girders. The use of a diagonal eye-piece is necesary for these steep sights.

(c) Many pointings on the targets are made to obtain mean positions of F and G.

(d) Repeat operations (b) and (c) but with the theodolite at C.

(e) The line joining the mean positions F and G on the shaft-top is then produced either by a fine wire under tension or by coplaning in both directions.

The Optical Transfer methods of correlation involve the use of such special theodolites as have auxiliary telescopes or have a tribrach with a central hole for transiting the main telescope for taking sights downwards do not give satisfactory results because of the short base line distance between two reference points sighted at the bottom of the shaft with the instrument set at the shaft-top. Similar the result in up-shaft sighting. The short base line rules out against optical correlation because for dissances less than 45 m the thickness of cross-hair gives too coarse a sight. It has been stated that the cross-hair covered 37.5 mm at 1050 m depth and 63 mm at a depth of 1950 m. Optical correlation carried out in stages of distances less than 45 m by repetition of the angles and using special target will also not give the desired accuracy as for example with an accuracy of 1 min (using wild theodolite fitted with an objective prism) per stage of 45 m, the cumulative error for a depth of 1000 m would be of the order of $20\sqrt{30}$ to $40\sqrt{30}$ or 110 to 220 seconds of arc. This materially exceeds the error with shaft plumbing method of correlation. Thus optical methods are not suitable for shafts exceeding 500 metres.

SHAFT PLUMBING FOR CORRELATION

In the majority of cases, correlation has to be carried out down one or more vertical shafts and this entails the suspension of plumb lines in the shaft or shafts.

It is desirable that the wire used in plumbing the shaft should be as fine as is consistent with its ability to support the plumb bob so that a minimum of surface is presented to outside

influences, e.g. ventilation and falling water, both of which might set up oscillations of the wire. Also, the smaller the diameter of the wire the easier it will be to make accurate bisection of it through the theodolite. The most suitable type of wire is hand drawn piano wire.

To avoid kinking, the wire may be wound on a small drum fitted with a brake or ratchet to hold it in any desired position. The plumb wire is lowered down the shaft with a small plummet about 2 kg weight attached to it . When it reaches the bottom the small plummet is replaced by a larger one, which may weigh upto 25 kg and the large plumb-bob may then be immersed in a barrel of water or oil to cut down the oscillation of the wire. The longer the wire is left in suspension it will reduce the oscillations and eventually come to rest. It may be necessary to raise the plumb-bob from time to time to allow for the stretch of the wire.

The wire should hang quite freely in the shaft and this can usually be ascertained from inspection by the surveyor travelling the shaft on top of the cage. In a shallow shaft a lamp can be passed slowly round the wire at the bottom and if the lamp can be seen continuously from the top of the shaft the wire is clear. Light washers may be passed through the wire at surface which will come down the shaft bottom indicating that there are no obstruction against which the wire is resting. The ventilating fan should be stopped during and at least one hour before the correlation.

To observe whether the plumb wire is at rest, it should be bisected by the cross-hairs of the telescope, the wtre illuminated by a lamp held behind it with a piece of white paper held between the lamp and the wire.

In a very deep shaft, it is possible that the wire will not come to rest and it is then necessary to determine its correct position. There are a number of ways of doing this–

(1) The amount of swing of the plumb-wire is observed by two theodolites set at right angles to each other. The telescopes are focused on the wire and the extremities of the swing are determined by means of scales placed at right-angles to the axes of the telescope and horizontally behind the plumb-wire. The position of rest can then be calculated for both directions from given number of observations of the amplitude of the oscillations of the plumb line. Theoretically, the amplitude should diminish at each oscillation but, in practice, it is found that the wire receives irregular impulses (ventilation, water, etc.) in addition to the regular impulse due to terrestrial attraction. Therefore, amplitude may not diminish.

The average of the observed amplitude will be used as the correct one for determining the position of rest providing that violent oscillations are disregarded.

Having calculated the position of rest, the telescopes are directed to the mean position on the scales, the wire adjusted to the cross hairs of the telescopes and the wire fixed in that position.

(2) In this method scales are not used. The wire is followed by the telescope by turning the horizontal circle tangent screw to the end of the oscillation and the vernier is read in that position. The wire is then followed to its other extreme position and vernier read again. The difference between the two vernier readings gives the angle formed by the oscillation of the plumb wire. The mean of a series of such readings is taken to fix the position of rest of the wire. Simultaneous

observations with a second theodolite at right-angles to the first should be noted to fix the correct position of wire at rest.

(3) Some theodolites are fitted with a special diaphragm upon which fine horizontal scale has been etched in addition to the normal cross lines. The theodolite is so adjusted that the intersection of the cross lines is half way between the extremities of the oscillations, as indicated by a series of observations.

Plumb bobs are normally made of lead which has a high density and is unaffected by magnetic attraction. They should be symmetrical in order to aid verticality of the wire and should be fitted with vanes to dampen the twisting motion of the plumb bob when immersed in water or oil.

Shaft plumbing methods of correlation are laborious and time consuming requiring availability of the shaft to the surveyor for long periods of times, sometimes exceeding 24 hours, thus affecting adversely the hoisting of men, materials and mineral. It also becomes necessary to stop the main ventilator for the duration of the correlation survey so that the wires become reasonably stationary in as short a time as possible. The disruption and stoppage of the ventilation is particularly dangerous in deep and gassy coal mines where coal seams liable to spontaneous combustion are worked. Approval from the Directorate of Mines Safety would be required for stoppage of main fans in such mines.

Disruption of the wire planes also caused by such other factors as creep because of the weight of the plumb bobs, gravitational pull, magnetic and electromagnetic effects because of presence of steel shaft buntons, pipes, cables, etc. Even damping of oscillations by immersing the plumb bobs in oil or water does not help because very small lateral force, say due to dripping water or stray leakage ventilation currents is enough to displace the wire from their true position. A force equivalent to 2.72 gm is sufficient to displace a 600 m long vertical wire loaded at its lower end with a plumb bob weighing 136 kg by 12.5 mm of the vertical.

The methods of triangulation on moving wires are inaccurate because the individual wires do not behave as simple and identical pendulums. This has been observed by noticing that the periods of oscillations of the individual wires often do not agree with their theoretical values and moreover the perpendicular distance between adjacent wires on surface and underground often differs by more than their theoretical values, the underground distance sometimes even greater than the surface distance. The results of correlation surveys indicated that incase of two deep shaft mines there were differences in bearings of surface and underground coplane bases although the conditions were considered to be ideal, in case of a shaft 550 m deep the difference in bearings of surface and underground coplane was 2 min 34 sec, and in the other case a shaft depth of 810 m, the difference was 3 min 34 sec.– two wires having been used in each of the shafts.

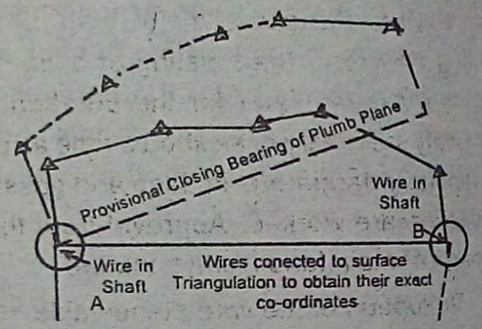
CORRELATION BY PLUMB LINES IN TWO SHAFTS

If both shafts are vertical, correlation may be effected by means of two plumb lines, one suspended in each shaft. The position of the plumb lines at the surface is incorporated to the surface triangulation and the length and bearing of the line joining the two wires (plumb-plane) is calculated in relation to Ordnance or an assumed meridian. A traverse is then made underground

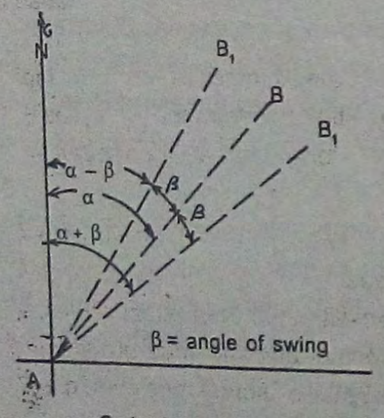
between the two wires and their co-ordinates calculated by reference to an assumed meridian. The bearing of the plumb-plane, as calculated from the surface co-ordinates of the wires is taken as its true value and compared with the bearing of the plumb-plane, calculated from the co-ordinates of the underground traverse. The difference in bearing is taken to be the error in direction of the assumed underground meridian, and the whole of the underground traverse has to be swung through this angle to bring the surface and underground plumb-planes into coincidence. If the surface co-ordinates of one of the wires is taken as the point of origin for the traverse calculation, the co-ordinates of the other wire, as obtained from the underground traverse, will not normally agree with its surface values. The underground traverse must therefore be adjusted so that this difference is eliminated.

There are two well-known **methods of swinging a traverse** from an assumed bearing into accordance with a known bearing :

(a) In the first method the angle through which the traverse has to be swung is applied as a correction to the bearing of each traverse line and the whole of the co-ordinates recalculated from the adjusted bearings.



Orientation by single wire in each of two shafts



Swinging a traverse

(b) The second method requires that the angle of swing should be very small, less then 20 minutes in which case the assumed co-ordinates can be adjusted to their grid values by an easily applied correction and the work of recalculating the whole of the traverse avoided. The principles of the second method are as follows :

In the figure, AB is the first line of a traverse, its length being x and its bearing α,

the latitude of point B = x cos α.

the departure of point B = x sin α.

If the traverse has to be swung through a small angle β then the true bearing of AB = α ± β, and the latitude of point B_1 = x cos (α ± β), and the departure of point B_1 = x sin (α ± β).

By expanding the equations,

Latitude of B_1 = x.cos α .cosβ ± x sin α sin β

Departure of B_1 = x.sin α . cos β ± x. cos α . sin β

But the latitude of B = x.cos α and its departure = x.sin α

Substituting these values in latitude and departure of B_1 –

Latitude of B_1 = lat B cos β ± dep. B sin β

Departure of B_1 = dep. B cos β ± lat B. sin β

If the angle of swing is very small cosβ may be taken as unity and sin β and β in radians will be practically identical, therefore

$$\text{Latitude of } B_1 = \text{lat B} \pm \text{dep B} \times \frac{\beta \text{ in sec.}}{246{,}265}$$

$$\text{Departure of } B_1 = \text{dep B} \pm \text{lat B} \times \frac{\beta \text{ in sec.}}{246{,}265}$$

It is quite a common practice to suspend a wire from the headgear platform, or through it if a small hole is drilled to accommodate the wire in position. The theodolite can either be set directly over the hole or the observation can be made from a satellite station. It may, however, be impossible to connect the wires directly and they may have to be connected to the nearest triangulation control by a precise traverse. These should be short and closed back on to their starting points so their accuracy can be judged and adjustments made.

The error in bearing of the plumb-plane below ground will depend upon the distance between the wires and their verticality. The closer together the wires the greater must be the precautions taken to ensure their verticality. The accuracy of this method of correlation is affected by errors (a) in the determination of the grid co-ordinates of the wires, (b) in the verticality of the wires and in the underground traverse. Mistakes and systematic errors in linear measurement and the errors in measuring those lines which run approximately at right angles to the plumb-plane must receive special attention.

Suitable checks will, to some extent, indicate whether the orientation conforms to the required limit of errors. An independent traverse might be made between the same pair of wires, preferably by a different route. In such a case the two bearings obtained for the plumb-plane should agree to less than 30 sec. of arc, and if both were of equal weight the mean of the two could then be taken as the most probable underground determination.

A rather more satisfactory check would be provided by having two wires suspended in one of the shafts, so that two plumb-planes would be available which could be connected independently and the bearings of a base common to each survey compared.

CORRELATION BY WIRES IN A SINGLE VERTICAL SHAFT

Where there is only one vertical shaft available for the work of correlation then the task becomes appreciably more difficult. Two plumb wires must be suspended in the shaft and the vertical plane connecting these two wires will be the plumb-plane. The azimuth of the plane must be determined. The width of this plane is very limited, about 6 m. Any lack of verticality will cause an appreciable discrepancy in the bearing of the plumb-plane. The discrepancy in verticality will increase with depth and therefore the deeper the shaft the more care will be required in plumbing. If the plumb wire will not settle then observations of the mean position will have to be taken and the wire clamped. In deep shafts several plumb-planes in the one shaft may be used in an effort to minimise the error due to discrepancies in verticality.

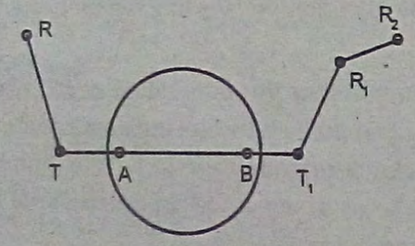
COPLANING OR EXACT ALIGNMENT METHOD

In this method the object is to place the axis of a theodolite exactly in line with the plumb-

plane. The theodolite on the surface will be exactly aligned and by observing angles between the plumb-plane and nearest triangulation station the azimuth of the plumb-plane can be determined. The theodolite must be set up close to one wire, about 3 to 3.5 m away aligned firstly by eye. If the telescope is closer then both wires can easily be seen by the change of focus. The wires are distinguished by the fact that the nearer wire is the thickest. After aligning the tripod by eye the instrument is attached and accurately levelled. The two wires will be observed. The telescope is pointed at the first wire. The nearer wire is then brought into focus and if the theodolite is out of line will be apparent in that the wire and the vertical cross-hair will not be coincident. A scale can be placed behind this wire and the approximate amount of movement of the theodolite can be caloulated. Another method is to align by trial and error.

The modern theodolites are fitted with tribrach heads which have screws incorporated so that lateral movement can be attained without disturbing the level or the normal shifting head. If the theodolite in use is not equipped in this way then the tripod head should be levelled as accurately as possible before alignment. This method requires a good deal of patience and skill. It must be remembered that the movement required by the theodolite is in the direction opposite to that apparent through the telescope.

When both wires are in line viewed through the telescope, which means that the line of sight lies in the plumb-plane. This must be transferred to the underground reference stations. This can be done by measuring the horizontal angle BT_1R_1 from the co-planed station below ground. The plumb-plane azimuth is determined on the surface and the azimuths of the underground lines can be calculated. The length of the lines can be measured and the co-ordinates of the station determined.

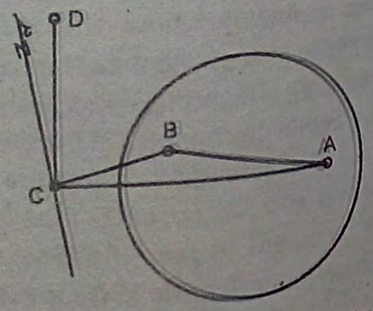


Coplaning Method

WEISBACH TRIANGLE OR APPROXIMATE ALIGNMENT METHOD

The method involves setting up the theodolite slightly to one side of the alignment position so that its line of collimation is almost in line with the plumb-plane, the angle of deviation being only of the order of one or two minutes.

The arrangement of Weisbach triangle in its relation to the line CD joining two stations in the surface survey has been shown in the figure. The azimuth of CD being known, the azimuth of the plumb-plane AB may be calculated, provided that the lengths of the sides of triangle ABC, the angle DCA and also the small angle BCA are accurately measured.



Weisbach Triangle Method

The sides of the triangle ABC must be measured very carefully with a tested steel tape.

When the angles at A and C are very small, the sum of the lengths of the two short sides of the triangles is virtually equal to the length of the longest side, the difference being so small as to be scarcely measurable.

The small angle at C, the weisbach station, may be measured as follows : Set up the theodolite at C and carefully make all temporary adjustments. Direct the telescope to station D and then observe the angle DCB in the usual manner, taking a number of readings of both verniers on each face of the instrument. The mean value is the correct value of the angle DCB. In a similar manner, obtain the mean value of the angle DCA. The dirrerence between the two angles so obtained is the correct value of the angle BCA.

The triangle ABC now be sloved and the small angle at A, namely CAB, calulated using the Sine Rule.

In the triangle ACB, $\text{Sin } A = \frac{a \times \text{Sin } C}{c} = \frac{BC \times \text{Sin } C}{AB}$

Since the sines of very small angles are porportional to the angles themselves and thus it is permissible, as the angles at C and A are very small to state–

Angle A (in seconds) $= \frac{BC}{AB} \times$ angle C (in seconds).

The **azimuth of BA** = azimuth CD + angle DCA + angle BCA

Underground, the procedure is similar except that, in this case, the azimuth of the plumb-plane BA is known and the bearing of the underground base line is unknown and must be calculated.

The distance betwen the weisbach station and the nearer wire should be as short as possible which will normally be governed by the focussing of the telescope.The plumb-plane BA must be as wide as possible. The width of the plumb-plane will be entirely dependent upon the diameter of the shaft, the position of cages and shaft fittings. Since the calculated angle is a function of the ratio between CB and BA, it will be affected by errors in the measurement of these two distances. As the angles involved are, however, very small, slight errors in CB and BA are not important and it is unnecessary to measure them with more than ordinary care.

Although it lacks the simplicity of co-planing, this method, if carefully and systematically applied, is no less accurate. It imposes less strain on the observers. The choice between the co-planing and weisbach triangle methods in single shaft correlation is a matter of personal preference and, if time permits, it would be sound practice to use both methods.

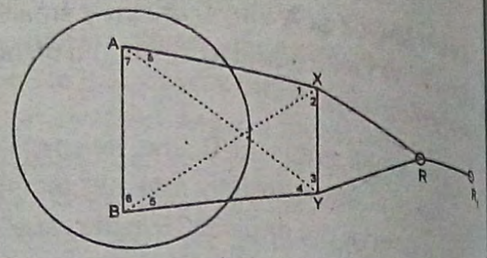
WEISS QUADRILATERAL METHOD

If the layout of the shaft, either at the surface or underground, precludes exact or approximate alignment, the method of *Weiss quadrilateral* provides a possible alternative.

The weiss quadrilateral is formed by the two wires and the two instrument stations. Theodolites are set up at station X and Y and the angles 1, 2 and YXR are measured at X and the angles 3, 4 and XYR are measured carefully by constant repetitions. The lengths of the sides of the quadrilateral and the diagonals are measured as a check on the angles although all the

calculations involve the angular measurements.

The angles at R, YRX, XRR, and YRR, are measures Sufficient informations have now been obtained to calculate the azimuth of the plumb-plane at the surface or the azimuth of RR, below ground.



Weiss Quadrilateral

To calculate the angles at A and B

$5 = 180^0 - (2 + 3 + 4)$

$8 = 180^0 - (1 + 2 + 3)$

It is known that the product of the sines of the left-hand angles equals the product of the sines of the right-hand angles in any polygon.

$$\text{Therefore } \frac{\text{Sin } 7}{\text{Sin } 6} = \frac{\text{Sin } 2.\ \text{Sin } 4.\ \text{Sin } 8}{\text{Sin } 1,\ \text{Sin } 3,\ \text{Sin } 5}$$

Since all the angles on the right-hand side are known then $\frac{\text{Sin } 7}{\text{Sin } 6}$ has a value say x

But 7 + 6 = 2 + 3 which in known value, say θ

Therefore, 7 + 6 = θ and hence 7 = θ - 6

$$\text{Since } \frac{\text{Sin } 7}{\text{Sin } 6} = x, \text{ then Sin } 7 = x \text{ Sin } 6 \text{ and Sin } (\theta - 6) = x \text{ Sin } 6$$

Expanding Sin θ cos 6 - cos θ Sin 6 = x sin 6.

Sin θ cos 6 = x sin 6 + cos θ sin 6 = sin 6 (x + cos θ)

$$\text{Therefore } \frac{\text{Cos } 6}{\text{Sin } 6} = \frac{x + \cos\theta}{\text{Sin } \theta} \quad \text{or, cot } 6 = \frac{x + \cos\theta}{\text{Sin } \theta}$$

Angle 6 can thus be calculated and angle 7 found by subtraction. Thus the azimuths of the line are determined and eventually the co-ordinates of stations. Observations may be made from both sides of the shaft if possible. Several plumb-planes may also be used.

Errors of Correlation By Wires

The correlations of the surface and the underground workings carried out by a single wire in each of two or more shafts or two or more wires in a single shaft shall be deemed to be within the required limit of error when the difference in values of the azimuth of any reference line of the underground survey, relative to the surface reference base line, as determined by two or more independent series of observations between wires, does not exceed two minutes of arc.

PRECISE MAGNETIC CORRELATION

In this method the magnetic bearings of surface and underground base line are determined and then a travrse is carried out from each to a single plumb wire suspended in the shaft. The magnetic azimuth of each line is determined and the difference applied to the grid bearing of the surface base will give the grid azimuth of the underground base. The traverse will give the co-ordinates of the underground stations. The method uses the tubular compass attached to the theodolite to dertermine the magnetic bearings of the lines. Two theodolites will be used, one on the surface

and the other underground. The location of the surface base line and the underground base line have got to be chosen with great care. If it is at all possible the surface base line should be vertically above the underground base. The stations were the observations are to be taken must be free from local disturbances. On the surface it is fairly easy to see whether there are disturbing influences in the locality. In underground working it is sometimes very difficult to find a road which is free from local attractions due to pipe lines, power cables and use of steel below ground. It may be that a special heading will have to be driven supported solely by wooden supports before a station can be made. This underground station must be tested to ensure that it is free from disturbing influence before correlation is attemped.

The time of observation should be chosen carefully because of the variation of the magnetic needle. The week before and week after magnetic storms should be avoided.

The two theodolites are set up on the stations and magnetic observations must be taken simultaneously on the surface and underground. The observations are repeated at fixed intervals. Graphs are plotted of the observations. The following day the positions of the two theodolites are reversed and the observations repeated. The theodolite traverse to the plumb wire in the shaft is then carried out. If two base line can be established underground and their respective azimuths found and then a theodolite traverse is carried out between these two underground base lines it will check the work of the corrections.

The advantages of this method are that if there are several base lines in different levels these can all be oriented with little trouble and also this method entails little or no disorganisation of the workings. If sufficient care is taken this method will give fairly good results.

CORELATION WITH GYROTHEODOLITE

The need for some other method of correlation had long been felt because of the difficulties and limitations of different methods such as wire correlation, magnetic methods etc, which are laborious, time consuming, requiring availability of the shaft to the surveyor for long periods and dusruption and stoppage of ventilation. Mine correlations by using gyroscopic instruments have recently been carried out much more speedily and with better degree of accuracy even in very deep mines. The gyroscopic instruments direction of an underground line situated anywhere in a mine can be determined with an accuracy of about 12 seconds of arc and with further modification and refinements of the instruments still better result can be expected.

Establishment of a Surface Gyro Base : It is generally expendient to establish a gyro base on the surface of a mine for the purpose of calibrating the instruments and for providing a reference base for drifting the orientation of surface surveys. The base should not be less than 300 m long., and the end stations be carefully chosen so as to clear of any likely subsidence movements. They should also be in an area where the line connected them is not likely to be obstructed by other mining operations or the growith of tress or the construction of buildings or power lines, etc. They should be as far away as possible from high tension electric cables. The line, if possible, should run in a north south direction for practical convenience through this is of

little consequence. The end station should be permanently marked by constructing cement concrete pillars or by driving in metallic piles.

Co-ordinates of end stations should be determined by the usual surveying technique of traversing or triangulation and adequately checked against mistakes. By calculating the meridian convergences at both the end stations, the azimuth of the line should be determined in both directions. If possible the azimuth of the line should also be established by astronomical observations. This will afford a further check on this important operation and a sound gyro base on the surface will thus be established.

Establishment of Gyro Base Underground : The gyro base underground should be carefully selected in such areas as are free from strata movements and as far away from electric cables or electrical disturbances. The area should also be, if possible, in a quite portion of the mine free from excessive traffic movement. A base line 120 to 180 m long will suffice, and a base of less then 50 m in length should not be used. The end stations should be free from stray ventilation current unless optical plumbing devices are available both for the gyro instrument station and for the target at the other end of the base. The end station should be marked by permanent markings preferably in the roof.

The survey is started by calibrating the instrument at one of the end stations of the surface base whose azimuth has been precisely determined earlier. Any of the methods of observations can be used, but which ever method is used the same method should be used in all the subsequent observations in the survey. Though few observations are sufficient, it is preferable to take eight observations to detect any drift in the gyro element. After completing the first calibration survey on surface base, the instrument should be shifted to the underground base and in case the instrument is non-flame proof a check for the presence of explosive gases be carried out . The instrument be set at one of the station and oriented by quick method in the direction of the azimuth approximately. The correct azimuth should be determined by the same observation procedures as on the surface, taking the same number of observation. The instrument should then be set up at the far terminal of the base, and the azimuth determined in the reverse direction. A rough calculation will soon determine whether the two results are sufficiently consistent.

The instrument should then be brought back to the surface base and recalibrated along the same line as before but preferably in the reverse direction. The azimuth of the line (surface base) in two opposite direction be determined from the known grid azimuth of the base and the calculated values of meridian convergences at each terminal. The instrument constant '*azimuth of gyro indicated north*' should be determined from the azimuths of the base line in two individual value agree sufficiently closely.

The angle between the gyro-readings and the readings to the other end of the underground base should be calculated for the observations at each end of the base. The instrument constant determined on surface should be added to this angle to give the two underground azimuths. The meridian covergence for each of the gyrostation should then be computed and applied to the previously applied azimuth to give the corresponding grid azimuth. The mean of the two azimuths (apart from the 180^0 difference) should then be adopted to give the final result provided again that the two individual results agree sufficiently well with each other.

Transference of co-ordinates, if necessary, can be done through the shaft by either suspending a single plumb wire or by usuing the recently developed electro-magnetic distance measuring instruments like the Geodimeter, the Distomat etc. The latter methods are quite speedy and give results of an accuracy compared to that with other methods.

Precautions when Using Gyrotheodolite

1. It is important to keep the converter in as cool a state as possible and in tropical climate the instrument should be protected during surface surveys by an umbrella, etc.

2. When taking instruments in windey conditions it is advisable to use an observation tent to protect the instrument.

3. The effects of magnetic field on preference of pendulum gyros do not appear to be dully understood. Some of the gyroinstruments available are fitted with magnetic shields and one manufacturer states that field greater than 2 oersteds will adversely affect their instruments. No mention is made of the influence, if any, of the direction of the field.

4. It is becoming a standard practice to omit the first one or two several circle readings when measuring the reversal observations obtained with the pennel MW 10 instrument.

Uses and Advantages : (a) Gyroscopic methods of correlation are being used in many other areas besides mine surveying : Construction engineers who drive tunnels through mountains or below the rivers and highways find gyroscopic methods much more conventional methods and in the long run less expensive too.

(b) Offset within an accuracy of ± 15 cm were determined using an instrument with an accuracy of ± 10 min of arc over a distance of 3 Km.

(c) Magnetic declinations can be accurately determined using a gyrotheodolite provided with a maginetic compass.

(d) The correlation survey carried out by conventional plumb-wire method and subsequent checking by gyrotheodolite, the agreement between the azimuth bearings was of the order of 12 seconds of arc.

(e) A gyrotheodolite can save much time and moeny in a correlation survey. The cost of the instrument can be recovered in a very short period of time.

QUESTIONS & ANSWERS

Question 1. What are the purposes of correlation surveying in mines ?

Answer : PURPOSES OF CORRELATION SURVEYING

The various purposes of correlation of surface and underground surveys are –

1. To determine with high degree of accuracy the relative positions of the underground roadways and faces may be correctly laid down on the working plan.
2. To fix the boundary of the mine for limiting the extent of underground working.
3. To fix the positions of railways, roads and important buildings or structures on mine

plans so that statutory restrictions imosed may be strictly followed during workings under them.

4. To mark the river, lake, tank, pond, etc. on mine plans so as to take adequate precautions against inundation.

5. To select suitable site for proposed pits, drifts, or boreholes to connect with underground workings.

Question 2. What is correlation survey ? Enumerate the different methods of correlation of surface and underground surveys. What should be the accuracy in such surveys ?

Answer : CORRELATION SURVEY

Correlation, in surveying, is the method of surveying by which the surface survey and underground survey are connected to the same base.

Different Methods : The methods employed for correlation surveys are controlled to a large extent by the conditions at the mine. The different methods are–

1. Direct traversing when workings are reached by adits or drifts.
2. Shaft plumbing methods when access is by means of vertical shafts–
   (a) One wire in each of two shafts.
   (b) Two or more wires in a single shaft by–
      (i) Coplaning or exact alignment method;
      (ii) Approximate alignment or method of weisbach triangles;
      (iii) Methods of weiss quadrilateral.
3. Optical method or direct transference of azimuth down a shaft by a transit theodolite.
4. Magnetic methods–
   (a) By tubular compass.
   (b) By magneto theodolite.
5. Gyro–Theodolite method.

Accuracy : The permissible angular error for correlation is ± 2 minutes of arc. Therefore, the accuracy in correlation surveys should not exceed the maximum permissible angular error.

Question 3. Describe the coplaning method of correlation survey for surface and underground workings of a mine. What precautions will be essential for attaining high precision in this type of work.

Answer : THE COPLANING METHOD

The coplaning of direct alignment method consists of setting up theodolites in the plane of the two plumb lines in one shaft at the surface and underground and thus transfering the bearing of surface survey directly belowground.

Instrument Required :

1. Two hard drawn piano wire each 0.1 cm diameter having breaking strength of 150 kg and of sufficient strength to cover the shaft.

2. Two small drums fitted with brake or ratchet.
3. Two biconical wooden plummets weighing 2 kg. each.
4. Two symmetrically shaped lead plumb bobs weighing 25 kg. each and fitted with vanes to dampen the twisting motion when immersed in water.
5. Two small pulleys, tin washers, buckets of water.
6. A theodolite preferably microptic type with telescopic legs.
7. Steel tape, thermometer, spiring balance.
8. Scale for determining mean position of wires.
9. Field book, pencil, oiled paper, safety lamps, pegs, nails, paints, etc.
10. Telephone or other means for communication of messages from surface to underground and vice versa.

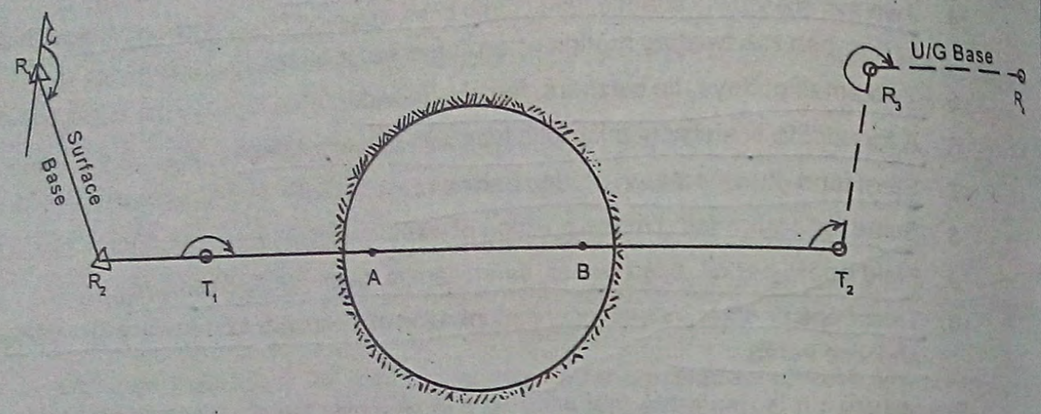
Procedure : It is desirable that wire used in plumbing the shaft should be as fine as is consistent with its ability to support the plumb-bobs so that a minimum of surface is presented to outside intluences e.g. ventilation and falling water, both of which might set up oscillation of the wires. Also, the smaller the diameter of wires the easier it will be to make accurate bisection of them through the theodolite.

To avoid kinking the wires are coiled on small drums fitted with brake to hold them in any desired position. The plumb wires are lowered down the shaft through small pulleys on the headgear as far apart as possible with small wooden plummets attached to the wires. When the wires have reached the bottom of the shaft light tin washers are passed down the wires from surface to check that the wires hang free from obstruction or kink. The smaller plummets are then replaced by heavier plumb-bobs made of lead to eliminate possibility of magnetic attraction. The plumb-bobs are immersed in buckets of water to cut down the oscillation of the wire due to (i) irregular mass in strata, (ii) simple pendulum motion, (iii) air current, and (iv) dropping water. The longer the wires are left in suspension it will reduce the oscillations and will eventually come to rest. It is necessary to raise the plumb-bobs from time to time to allow for the stretch of the wires. As the wires are affectd by the ventilation current, the fan should be stopped during and at least one hour before commencing the survey operations.

From the surface traverse, two stations R_1 and R_2 are fixed as the surface base. The theodolite is set at station T_1 marked on the surface pit bank near the shaft judged by naked eyes in alignment with the two wires. The station T_2 should be as close to the nearest plumb wire as good focussing will allow so that both wires can easily be seen by change of focus. The wires are distinguised by the fact that the near wire is thickest. The instrument with the vernier at zero is manoeuvered by means of its sliding head until it is in exact line with the wires. The angle $A T_1 R_2$ is measured. Then the theodolite is set up at station R_2 and the angle $R_1 R_2 T_1$ is measured. Distances $R_2 T_1$, $T_1 A$ and AB are measured very precisely. The azimuth of the plumb-plane AB and the co-ordinates of A and B with respect to surface base station $R_2 R_1$ are calculated.

Similarly the theodolite is set at station T_2 belowground in exact alignment with the plumb-plane AB and the angles $B T_2 R_3$ and $T_2 R_3 R_4$ are measured. Distance $B T_2$, $T_2 R_3$ and $R_3 R_4$ are

also measured. A number of roof stations are fixed on the underground base line R_3 R_4 for

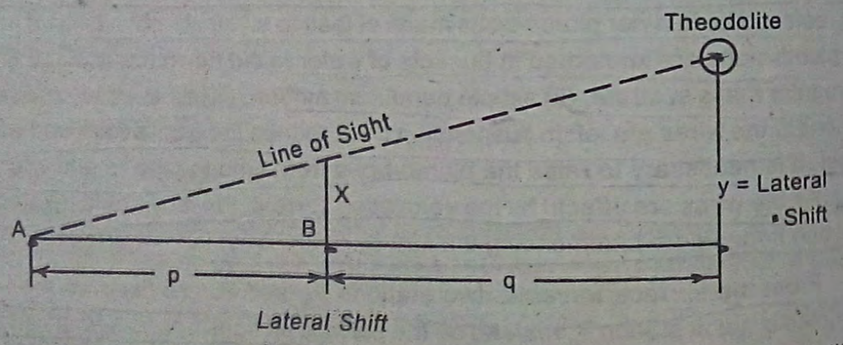


Coplaning Method

subsequent underground surveys. The azimuth of the underground survey lines and the co-ordinates of the underground stations are calculated from the azimuth of the plumb-plane AB and the co-ordinates of the plumb wires A and B which have already been established at the surface.

Lateral shift of instrument : The question of lateral shift or the distance by which the theodolite is to be shifted laterally for exact alignment may arise.

If the distance between A and B be 'p' and that between B and the theodolite station be 'q', the station A is sighted and with a plotting scale at B the displacement 'x' of wire A from the line



Lateral Shift

of sight is found out, and from the principle of similar triangles, lateral shift of the theodolite,

$$= \frac{x\,(p + q)}{p}$$

After completion of the survey for details within the surface triangulation frame work and also the underground traverse survey, the two surveys are now treated as one survey with a common plumb-plane and calculation and plotting should be done accordingly.

Precautions : The following precautions are to be observed –

1. The most desideratum in plumbing in a shaft with two wires is that the distance between them should be as great as it is convenient to make it. Since the error involved in determining the

position of the wires is probably in the inverse ratio to the distance between the wires, the initial error should be reduced to a minimum by suspending them as far apart as possible.

2. Lead or brass plumb-bobs must be used in plumbing a shaft to eliminate magnetic attraction in the vicinity of which there may exist a considerable mass of some magnetic rocks in which case the distance between the wires at the bottom of the shaft may not agree with the distance measured at the surface.

3. The plumb-bob must hang freely and must be free from kinks.

4. They must not stick against sides of water bucket, they must hang freely under water.

5. Air currents should be minimised by stopping the fan during and at least one hour before the observations.

6. Verticality of plumb wire must be ensured. A third wire may be suspended in the shaft as a check on the verticality of the wires, and in such a case all three wires should be carefully aligned at the surface and underground alignment should be checked.

7. In coplaning method the length of the line from the instrument station to the nearest wire must not be less than the distance between the two wires and must not be greater than twice that length.

8. The lines T_1 R_2 and T_2 R_3 should be as long as possible.

9. Plumb wires should be sighted against a white background oiled paper, illuminated from behind for facilitating observations.

10. For rapidity of work, it being difficult to steady the wires, the mean position of swing should be observed by means of a scale behind each wire, or by using a diaphragm with fine scale.

11. The permanent adjustments of the theodolite must be checked before the survey work.

12. All sights should be taken using both faces of the instrument. All angles should be very carefully measured by repitition method, at least six readings should be taken at each station and then averaged.

13. The distance between the wires at the pit bottom should be carefully measured and checked with the surface measurement. All other distances at surface and underground must also be measured with extreme care.

14. If possible observations should be made from both sides of the shaft.

Question 4. Write short note : Weisbach Triangle.

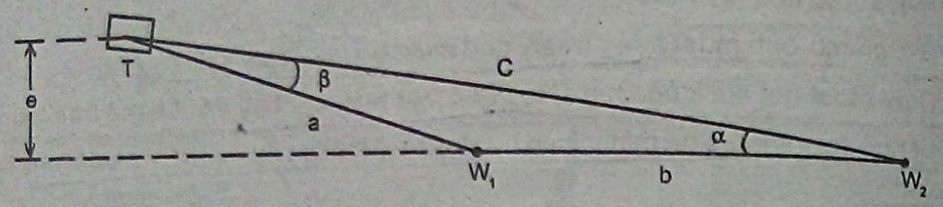
Answer : WEISBACH TRIANGLE

When transferring ground level azimuths underground by suspensions of two wires down a shaft it is difficult to set up the theodolite exactly in line with them. In addition, one wire being nearer obscures the wire beyond, leading to an inaccurate bisection. In Weisbach's method the theodolite is set up slightly out of line forming a small triangle with the two wires. This triangle is known as *Weisbach's Triangle* and the azimuth of the line joining the two wires is found by solution of the triangle so formed.

α is obtained from the triangle $W_1 TW_1$ by the sine rule :–

$$\text{Sin } \alpha = \frac{a \text{ Sin } \beta}{b}$$

and, $e = c. \sin \alpha$



Weisbach Triangle

In a weisbach triangle, the angle $W_2 TW_1$ called the Weisbach angle β and the ratio $\frac{c}{b}$ must be very small for which the theodolite station (T) should be very near to the wire (W_1). If the weisach angle is less than 25 minutes, there is no necessity of measuring sides precisely but the angle should be measured very accurately to avoid any error.

Question 5. Describe a method of connecting the surface survey with the survey of underground workings of a mine when only one shaft is available for survey work. Explain the precautions to be taken in work.

Answer : WEISBACH TRIANGLE METHOD OF CORRELATION

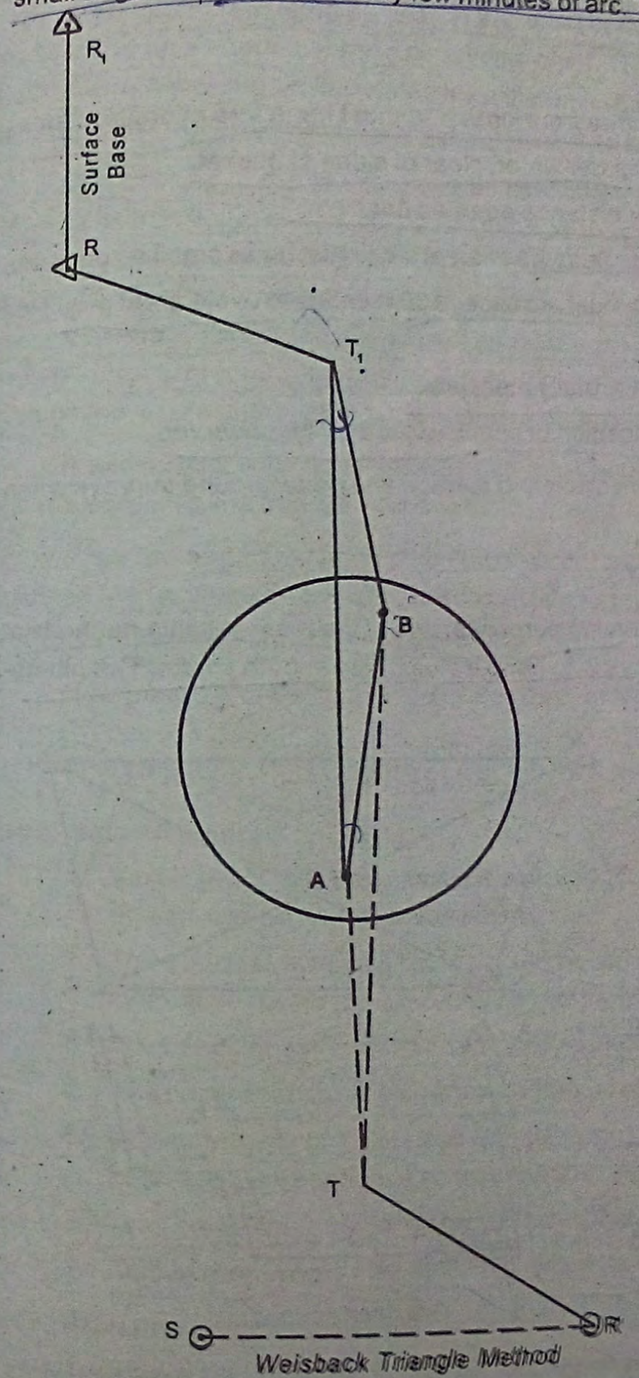
Out of the different methods, the Weisbach method of correlation is usualy prefered when only one shaft is available. The method consists of suspending two plumb wires which form a plumb plane and azimuth of the same is determined at the surface and taken underground at the shaft inset and thereafter transferred to the underground base or reference line.

Two piano wires A and B, 18 S. W. G. made of anticorrosive crucible steel 0.1 cm diameter having breaking strength of 150 kg and to avoid kingking coiled on small drums fitted with brake or ratchet to hold them in any desired position, are lowered down the shaft by means of biconical wooden plummets weighing 2 kg. each. When the wires have reached the bottom of the shaft tin washers are passed down by the wires from the surface to see that they are free of obstruction. The smaller plummets are then replaced by symmetrically shaped finned plumb bobs made of lead weighing about 25 kg. When the wires hang vertically each plumb bob is immersed in a barrel of water to cut down oscillation due to (i) simple pendulum motion, (ii) irregular mass in strata, (iii) air current, and (iv) dropping water. The longer the wire is left in suspension it will reduce the oscillation and will eventually come to rest. It may be necessary to raise the plumb bob from time to time to allow for the stretch of the wires.

From the surface traverse, stations R and R_1 fixed as surface base RR_1 whose aximuth is very carefully determined.

The theodolite is set at T_1 at surface as closed to the near wire B as focussing will allow

and almost in line with the plumb-plane AB produced and connecting to the surface bese. The small angle BT_1A should be only few minutes of arc. The triangle ABT_1 is known as the "Weisbach triangle".



Weisback Triangle Method

To measure the small angle BT_1A, the theodolite is set up at T_1 and all the temporary adjustments are made. The telescope is directed to wire B and then the angle BT_1R is observed in the usual manner, taking at least three rounds of readings on each face of the instrument. The mean value is the correct value of the angle. In a smilar manner, the value of angle AT_1R is obtaied and the difference between the two angles thus obtained is the correct value of the angle BT_1A.

The measurements of the sides of the triangle ABT_1 are made with a steel tape. At least six readings of each side should be taken, the tape being held quite level and fully supported, the average value being used as the correct one.

The theodolite is next set up at R and the angle R_1RT_1 is observed and the distances RR_1 and RT_1 are also carefully measured.

To calculate the azimuth of the plumb-plane AB, the small angle T_1AB must be calculated. The triangle T_1AB :–

$$\frac{BT_1}{\text{Sin } A} = \frac{AB}{\text{Sin } T_1}; \text{ or, Sin } A = \frac{BT_1 \text{ Sin } T_1}{AB}$$

The sines of very small angles are proportional to the angle themselves and therefore

$$A \text{ (in seconds)} = \frac{BT_1 \times (T_1 \text{ in seconds})}{AB}$$

The azimuth of the plumb-plane having obtained at the surface, a series of similar observations are taken belowground with the theodolite at T and so having calculated the angle

TBA at the shaft bottom, the azimuths of AT and BT can be calculated. The angles ART' and BTR' have also been measured and so the azimuth of the line TR' can be calculated. Finally be measuring the angle SR'T the azimuth of the underground base line RS can be deduced.

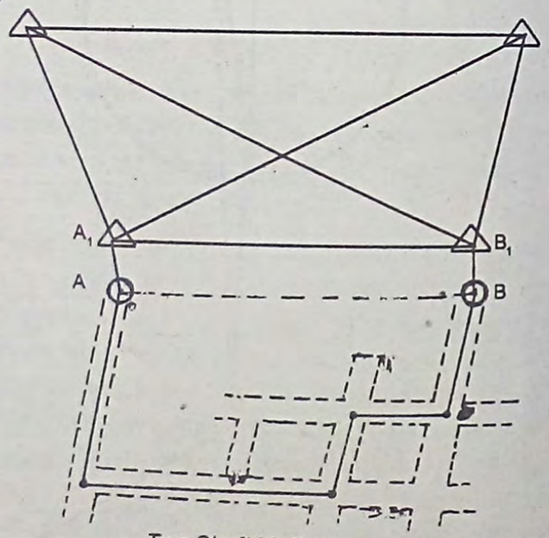
PRECAUTIONS REQUIRED

1. The plumb wires must be vertical, free from kinks and must hang free of obstructions.
2. The plumb bobs must hang freely under water, clear of sides of bucket.
3. The plumb wires must be as much distance apart as possible.
4. The distance between the instrument and the nearest wire must be as small as possible.
5. The weisbach angles BTA or BT, A must not exceed 30 minutes to avoid great precision in the measurement of sides.
6. The weisbach angle must be very carefully measured.
7. To ensure accuracy two or more position of plumb wires may be observed.

Question 6. Describe briefly the method of correlating surface and underground surveys when two shafts are available.

Answer : CORRELATION BY TWO SHAFTS

When two shafts are available for surveing purposes, one of the most suitable methods of correlation surface and underground surveys is to hang a plumb-line in both shafts. The plumb-line form the terminal stations of the surface and underground surveys, and consequently when the surveys have been completed their relative positions may be determined by calculation or by orientation of the plane of undergtound workings upon the plane of the surface, when these are plotted to the same scale.



Two Shaft Method

The main steps in the process of connecting the surveys by these methods are as follows :

1. To make a triangulation survey at the surface to form an accurate frame work for the detailed survey of the surface.
2. To suspend a wire in each shaft.
3. To locate the positions of the wires accurately at the surface by connecting them to the surface triangulation so that the length and bearng of the line joining the two wires and the co-ordinates of the wire stations may be computed.
4. To make an accurate underground traverse between the two wires.

Let A and B represent the plumb lines in the two shafts. New triangulation stations A_1 and B_1 near the mouth of each shaft, are established which are connected to the main triangulatic

scheme and also to the plumb-lines by measuring the angles and distances precisely. The co-ordinates of plumb-plints A and B and also the length and bearing of AB are then calculated.

A traverse is made underground from wire to wire along the roadway connecting the two shafts using an assumed or false directional base. The co-ordinates of second wire B relative to the first wire A are aagain calculated from the underground traverse, and the length and also the bearing of the plumb-plane AB joining the two wires computed.

The bearing of the plumb plane as calculated from surface co-ordinates of the wires, is taken as its true value and compared with the bearing of the plumb-plane calculated from the co-ordinates of underground traverse. The length of AB as computed from underground traverse should agree closely with that obtained at the surface, but the bearing will differ owing to the assumption of a zero base direction. The difference in bearings is the error in direction of the assumed underground meridian, and the whole of the underground traverse has to be swung through this angle to bring the surface and underground plumb-planes in coincidence.

Angle through which the traverse had to be swung is applied as a correction to the bearing of each travaerse line and the whole of the co-ordinates recalculated from the adjusted bearings.

If the amount of swing is small (less than 1 degree), the assumed co-ordinates can be adjusted to true value be applying the correction formula, and the work of recalulating the whole of the traverse avoided :

$$\text{New Latitude of Bx} = \text{Latitude of B} \pm \text{Dep. of B} \times \frac{\text{Amount of swing in seconds}}{206{,}265}$$

$$\text{New Dep. of Bx} = \text{Departure of B} \pm \text{Lat. of B} \times \frac{\text{Amount of swing in seconds}}{206{,}265}$$

Precautions Required

1. Verticality of the wires, must be ensured. The wires must hang free of any obstruction and should be at rest or almost so.
2. The shortest available route should be followed for the underground traverse. If two alternative routes are available, both of them may be used for a check upon each other.
3. The conditions of underground traverse should be good.
4. The number of drafts belowground should be reduced to a minimum.
5. Measurements of angles and distances must be very carefully done. Checks should be obtained by repetition of some method or by any alternative method.
6. True co-ordinates of wires on surface should be accurately determined.
7. Two or more wires may be used in each shaft for ensuring double check.

Advantages :

1. This is an ideal method of correlation. This is positive in as much as two plumb-lines form physical points of contact between both the surface and underground surveys.
2. If the two shafts are reasonably far apart, the resultant base line is of sufficient length to ensure the requisite degree of accuracy without excess precautions for the angular measurements.

Disadvantages :

1. The surveyors must have posessions of both shafts for a period of about two days to enable the wires to be suspended and for carrying out necessary survey works.
2. Problem may arise in running the cages due to the obstruction of suspended wires.
3. In deep shaft it is difficult to bring the wires to rest because of the air currents, droppping water or simple pendulum motion.
4. The traverses between the two shafts, both on the surface and belowgroud, may be difficult due to unavoidable local conditions in regard to both the length and the number of the sights.
5. In deep shafts far apart, covergence of the verticals makes an appreciable difference to the wire interval belowground as compared with their positions at surface.

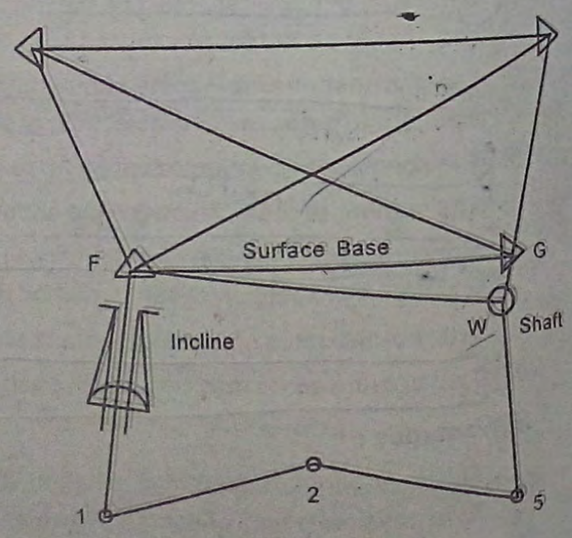
Accuracy : When correlation is effected by plumb-lines in each of two or more shafts, it is deemed to be within the required limit of error when the difference in the values of the azimuth of any reference line of the underground survey relative to the surface reference base line, as determined by any two or more independent series of observations between wires, does not exceed two minutes of arc.

Question 7. Describe a method of connecting the surface survey with the underground survey when one shaft and a drift is available.

Answer : CORRELATION THROUGH A SHAFT AND A DRIFT

An inclined shaft or a drift offers the most favourable condition under which it is possible to carry out the correlation of the surface and underground surveys.

The survey is commenced from one of the lines in the surface triangulation or from a course of draft in the traverse of the surface. Supposed one of the lines in the surface survey has been marked out in close proximity to the inclined entrance to the mine, the theodolite is erected and accurately centred over one of the line stations 'F' and having adjusted the verniers on the horizontal limb of the instrument to the bearing of the line, the telescope is directed to the other station, and then the vertical axis of the instrument is clamped in that position. A traverse survey is then carried out through the drift and the connecting roads extending from the bottom of that drift to the plumb-line 'W' which is suspended in the shaft. The survey is also carried out from the base-line to the plumb-line over the surface.



Correlation through a Shaft and a Drift

When the surveys are plotted on the plan surface will be shown in its true position relative to the workings of the mine.

All angles should be measured by repetition method, the distances being measured twice forward and at least once in the reversed direction. A sufficient number of permanent marks should be left to serve as reference lines for future under ground extensions.

A second angular check is provided by making a return traverse to the initial reference point G at surface through the drift so as to close the survey polygonally. If the co-ordinates of the wire in the shaft, as calculated from the underground survey, agree with the co-ordinates as calculated from the surface traverse, the connection of the surface and underground surveys may be said to have accurately established.

Question 8. Write in short a method of magnetic correlation for connecting the surface survey with the survey of underground workings of a mine. What are the draw-backs of these methods ?

Answer : PRECISE MAGNETIC CORRELATION

Principle : In this method the magnetic bearings of surface and underground base lines are determined and then a traverse is carried out from each to a single plumb wire suspended in the shaft. The magnetic azimuth of each line is determined and the difference applied to the grid bearing of the surface base will give the grid bearing of the underground base. The traverse will give the co-ordinates of the underground stations.

Procedure : In this method the tubular compass attached to theodolite is used to determine the magnetic bearings of the line. Two theodolites are used, one on the surface and the other underground. The location of the surface base line and two underground base lines have got to be chosen with great care. The surface base line should be vertically above the one underground base, if it is all possible. The stations where the observations are to be taken must be free from local disturbances. The time of observation should be carefully chosen preferably between 11 A.M. to 2 P.M. because of the variation of the magnetic needle. The week before and the week after the magnetic storms should be avoided. The observers must have reliable watches set at the same time.

The magnetic bearings of the surface base line and of one of the underground base lines are observed simultaneously at 5 minute intervals over a period of three hours from 11 A.M. to 2 P.M. The following day the position of the two theodolites are reversed and the observations repeated. The mean of the two days' observations would thus eliminate instrumental errors.

To arrive at the mean of a number of magnetic observations, it is not strictly correct to divide the sum of the bearings by the number of observations. The following formula gives the more accurate result :

$$\text{Average bearing} = \frac{a + 2S + b}{2(N-1)}$$

Where a = first reading

b = final reading.

S = sum of all intermediate bearings.

N = number of bearings.

A theodolite traverse to the plumb wire in the shaft is then carried out.

A similar set of magnetic observations are made at the surface base and at the second underground base on the third and fourth days and a theodolite traverse is carried in between the two underground base lines to check the work.

The result of each days' observations are plotted on separate graph tracing papers with vertical scale for the magnetic readings 10 minutes to 25 cms, and the horizontal scale half an hour to 25 cms. The underground curve is superimposed on the corresponding surface curve coinciding the hour lines so as to obtain the angle between the surface and underground bases irrespective of any diurnal variation that may have taken place. The results for each underground base are averaged so as to eliminate the effect of any constant difference between the instruments and the average for each base is taken as correct. Knowing the gird bearing of the surface base line and the angles between the surface line and underground bases, the grid bearings of the underground bases can be ascertained.

Finally, the difference of angle between the two underground bases is computed which must agree with the angle obtained by the underground traverse between the two bases, within two minutes or arc.

Advantages :

1. The magnetic correlation can be relied on to give accurate results when it is not feasible to adopt positive methods of correlation by means of wires, either because the shaft can not be made available, or because, the plumb lines can not be brought satisfactorily to rest or due to long traverse below ground in which the required limit of error is greatly exceeded or for some other local reasons.
2. The method entails little or no disorganisation of the colliery routine.
3. If there are several base lines in different levels these can all be oriented with little trouble.
4. If sufficient care is taken it gives fairly good result.

Drawbacks of Magnetic Correlation :

1. The accuracy of the magnetic method depends fundamentally on the ability of the magnetic needle to record quickly the variations of the magnetic meridian. An ordinary tubular compass gives good result in this respect, but on moderately quiet days only.

2. A minor irregularity in the magnetic needle itself affect the result seriously.

3. The results of the correlation are vitiated if the work is carried out on magnetically disturbed day. The magnetic storms repeat themselves about every 27 days and last for few hours or few days to produce magnetic disturbances to effect magnetic needles.

4. Beside the irregular variations, the secular and diurnal variations have direct influence over magnetic readings.

5. The stations where the magnetic observations are to be made must be free from local attraction. On the surface it is fairly easy to see whether there are disturbing influences in the locality. But in the underground it is very difficult to find a road free from local attractions due to

increased use of steel roof supports, and also for pipe lines and power cables.

6. It is desirable that the undergound base be sited vertically below the surface reference line, in order that their magnetic bearings maybe truly compared and to avoid any diffference of magnetic convergence. But it may be difficult to select an underground base line vertically below the surface base statisfying the other conditions e.g. free form local attractions.

7. There is possibility of change in the instrument constant during the period of transference from the surface to the underground base.

Question 9. Explain step by step the arrangements which you would make for proper suspension of the plumb bob from the shaft top to the pit bottom for correlation purposes. What step you would provide for obsevation of the plumb string in the shaft level if oscillations of the same could not be damped.

Answer : SHAFT PLUMBING FOR CORRELATION

Before proceeding to sespend the plumb lines in a vertical shaft, the top of the shaft should be covered over with substantial flooring of planks over steel joist to serve the two-fold purpose of preventing anything from falling into teh shaft, and checking the air currents in the shaft. Holes are bored in the platform vertically under the point of the plumb lines.

Wires used in plumbing the shaft should be as fine as is consistent with its ability of support the plumb bobs so that a minimum of surface is presented to outside influence e.g. ventilation, falling water, etc. which might set up oscillation of the wires.

To avoid kinking, the wires coiled on small drums fitted with brake or ratchet to hold them in any desired position. The plumb wires are slowly lowered down the shaft through small pulleys on the headgear as far apart as possible with small wooden plummet about 2 kg. in weight attached to the wires. When the wires have reached bottom of the shaft light tin washers are passed down the wires from surface to check that wires hang free from obstruction or kink. The smaller plummets are then replaced by heavier plumb bobs symmetrically shaped weighing 20 – 25 kg. each and fitted with vanes to dampen the twisting motion when immersed in water, made of lead to eliminate possibility of magnetic attraction. Plumb bobs should be raised or lowered from time to time to allow the stretch of the wires.

The plumb bobs are immersed in buckets of water to cut down the oscillation of wires due to (i) irregular mass in strata (ii) simple pendulum motion, (iii) air currents and (iv) dropping water. They must not stick against sides of water bucket and must hang freely under water.

As the wires are affcted by ventilation current, the fan should be stopped during and at least one hour before commencing the survey operations.

OBSERVATION OF PLUMB STRING IN SHAFT LEVEL

To observe whether the plumb wire is at rest, it should be bisected by the cross-hairs of the telescope, the wire illuminated by a lamp held behind it with a piece of tracing paper held between the lamp and the wire. If the oscillations of the wires could not be damped, it is then necessary to determine their correct positions. There are a number of ways of doing this –

(a) Schimdt Apparatus method
(b) L. D. Brown's Modification method
(c) Photographic method
(d) Roberts method
(e) Orange free state method
(f) By theodolites fitted with special diaphragm.

I would provide the following steps for observation of the plumb string in the shaft level if oscillation of the same could not be damped –

The amount of swing of the plumb string will be observed by two theodolites set at right-angles in each other. The telescopes are focussed on the wire and the extremities of the swing are determined by means of scales placed at right-angles to the axes of the telescope and horizontally behind the wire. The position of rest can then be calculated for both directions from a given number of observations of the amplitude of the oscillations of the plumb line.

The average of the observed amplitudes will be used as the correct one for determining the position of rest providing that violent oscillations are disregarded.

Having calculated the positions of rest, the telescope are directed to the mean position on the scales, the wire adjusted in the cross-hairs of the telescopes and the wire fixed in that position.

Question 10. What are the factors that will affect the choice of plumb wire and plumb bob for shaft plumbing ? What tests will you carry out to ascertain that the plumb wire is not entangled to any shaft fitting and it is vertical ?

Answer : SHAFT PLUMBING

The following are the factors that will affect the choice of plumb wire and plumb bob for shaft plumbing :

(1) **Material used for plumb wires :** Wires used for shaft plumbing may be of iron, steel, copper, brass, or phosphor bronze. In any case, the size of the wire should be as small as will safely carry the weight of the bob. The breaking stength of a wire limits the weight of the plumb bob which can be hung on it, while an increase in the diameter of a wire increases its resistances to air currents and, therefore, the possibility of deflection from the vertical. For long plumb-lines in shafts it may be piano-wire or rustless crucible steel wire. The former is very strong but is liable to the formation of kinks which are difficult to remove by weighting or other means. Crucible plough steel wire is more ductile and therefore more easily pulled straight by weighting, though its lower tensile strength necessitates a larger diameter of wire which is more difficult to bisect accurately. An estimate of the probable extension of the wire when weighted is however reaquired so that the weights shall hang freely in the receptacle containing them.

(2) **Diameter of Wire :** The diameter of the wire is important in two respects :

(a) the smaller the diameter the less will be the resistance to the air current, and (b) since we are obsertving the wire through theodolites from short distances then the smaller the diameter the easier it will be to make accurate bisection. The diameter of the wire will also affect the

breaking strain of the wire which, in turn, will affect the weight of plumb bob which can be suspended. It is desirabale that the wire used in plumbing the shaft should be as fine as is consistent with its ability to support the plumb bob so that a minimum of surface is presented to outside influences e.g. ventilation and falling water, both of which might set up oscillation of the wire. The most suitable type of wire for use as plumb bob wires is piano wire. This is cold-drawn wire with a high breaking strain so that smaller diameter of wire can be used.

(3) **Oscillation of Plumb Wires :** Although plumb wires are loaded, usually well towards breaking point, they are still affected by air currents in the shaft. Irregular mass in strata, simple pendulum motion and dripping water also cause oscillations of wires in a shaft. The movements of the wire caused by air currents can usually be reduced by alowing down or stopping the mechanical ventilation. Even so there will still probably be sufficient natural ventilation to affect the plumb wire causing the wire to hang in a parabolic arc. The amount of deflection tends to vary as the square of the depth and therefore, under similar conditions, a wire would be deflcected twenty-five times more at a depth of 1500 m than at 300 m. The plumb wire itself offers the greatest resistance to air currents and, since it is directly proportional to the diameter of the wire, a 20-gauge piano wire would tend to be deflected from the vertical twice as much as a 26-gauge. It is impossible to determine the amount of deflection in advance because of the unknown effect of the air currents,but a wire may hang perfectly vertical with a plumb bob of infinite weight. The weight of the plumb bob can be increased four-fold whilst the diameter of the wire, consequently, its resistance to air currents are only doubled. When, therefore, the air currents are very strong and can not be reduced it may be justifiable to use a heavy bob with a thick wire, but for most circumstances the lighter bob with a lower guage wire should be satisfactory.

To avoid kinking, the wire may be wound on a small drum fitted with a brake or ratchet to hold it in any position. The wire is lowered down the shaft with a small plummet attached to it. When it reaches the bottom the small plumment is replaced by a larger one which is immersed in a barrel of water or oil to cut down the oscillation of the wire. The longer the wire is left in suspension it will reduce the oscilations and will eventually come to rest. It may be necessary to raise the plumb bob from time to time to allow for the stretch of the wire.

In a very deep shaft, it is possible that the wire will not come to rest and it is then necessary to determine the average of the amplitudes which will be used as the corect one for determining the position of rest of the wire providing that violent oscillations are disregarded.

(4) **Choice of Plumb Bob :** Irregular-shaped weights are objectionable because in twisting they impart a wobbling motion to the wires, which adds to the difficulty in determining their mean position. Cast iron bobs have been suspected of being susceptible to magnetic influences, tending to deflect them from their ture position. Lead bobs are fully free from this suspicion. Plumb bobs for shaft plumbing are normally made of lead which has a high density and is unaffeced by magnetic attraction. They are as heavy as possible consistent with local circumstances and the strength of the wire. They should be symmetrical in order to aid verticality of the wire and should also be fitted with vanes to dampen the twisting of the plumb bob when immersed in water or oil.

To Ascertain the Plumb Wire Free From Obstruction : The wire should hang quite freely in the shaft and this can usually be ascertained from inspection by the surveyor travelling

the shaft on top of the cage. In a shollow shaft a lamp can be passed slowly round the wire at the botom and if the lamp can be seen continuously from the top of the shaft, the wire is clear. Light washers may be passed through the wire at surface which will come down the shaft bottom indicating that there are no obstruction against which the wire is resting. A freely syspended wire oscillates at a definite rate and the time for each swing (in one direction) is given by the formula-

$$t = \pi \sqrt{\frac{L}{g}} = 5.4938528 \sqrt{L}$$

Where, t = time required in seconds for one single oscillation (one way)

L = Length of pendulum (or wire) in metres.

g = 9.81

If the observed time for one oscillation agrees with the calculated time, this again is further confirmation that the wire is freely suspended.

Question 11. What are the advantages in correlating mine surveys and plans to the Ordanance Surveys to National Gird System ?

Answer : IN CORRELATING MINE SURVEYS TO NATIONAL GRID

Co-ordination of Mine & Ordnance Surveys : Underground and surface surveys of a mine are generally, oriented to a true meridian, magnetic meridian, or a local base line. Therefore, the boundary line between two adjacent mines may not necessarily be the same either in bearing or co-ordinate value. Further, the co-ord. of any common station may differ because the bearings and co-ordinates are originated from different initial stations and from different meridians. It is therefore, essential to provide a common reference throughout the country. This reference is known as National Grid. The positions of the centres of all mine shaft or other points of reference for underground surveys should be determined by reference to, and co-ordinated with the Ordnance Survey National Grid by a survey connecting such points with trigonometrical points of Ordnance Survey. There should be at least two such points of reference for each mine.

ADVANTAGES : (1) As both mine plans and Ordnance Survey maps are on the same scale, surface features to be shown on mine plans can be directly transferred from Ordnance Survey maps. (2) All detached surveys can be rigidly fixed to a framework fixed on a nationwide basis. (3) Where plans of adjacent mines are referred to a common National Grid, the accurate relative plotting of goaf lines on either sides of the boundary is facilitated. (4) Plans of abandoned mines can show the co-ordinates of points on the abandoned areas and these can be precisely plotted on the mine plans and necessary precautions can be taken beforehand when working are approaching the abandoned areas.

Question 12. Before commencing depilaring operation, it has been found necessary to accurately fix position of underground workings in relation to surface buildings and boundary of leasehold. Describe how would you carry out the survey.

Answer : CORRELATION SURVEY

The survey work consists of three main steps –

(a) Surface Survey, (b) Correlation Survey, and (c) Underground traverse

Surface Survey : The relative position of shafts, inclines, important buildings and structures, and also the boundary of leasehold should be determined accurately. For this purpose, a small triangulation survey comprising a series of interlacing triangles forming an accurate framework for the detailed survey is carried out. A base line and check base are measured by invar tapes and all necessary corrections are applied. Angles of each triangle are then measured precisely by a theodolite. Co-ordinates of all triangulation stations are then calculated and plotted on suitably gridded mounted paper. The meridian is also determined for the purpose.

A closed surface traverse is then run from triangulation stations to locate boundary of leasehold, important buildings, surface structures and other details to be shown on the plan in relation to underground workings. The survey after necessary checking and correcting are plotted on the triangulation framework.

Correlation Survey : Let the mine workings are approachable by a pair of shafts. The correlation of underground and surface survey consists of suspending plumb wires one on each shaft. The line joining the wires forms a common base for surface and underground surveys. The positions of the wires are precisely located at the surface by connecting them to the surface triangulation so that the length and bearing of the line joining the two wires and the co-ordinates of the wire stations may be computed.

Underground Traverse : A traverse is made underground from wire to wire along the roadway connecting the two shafts by the shortest route. Subsidiary surveys are then accurately carried out from the main theodolite traverse to locate the exact ends of drivages and the workings to be shown on the plan. Co-ordinates of underground traverse and subsidiary survey stations are then calculated and the details plotted on the same mounted paper using different colour. In this manner the positions of important surface structures, etc. in relation to underground working may be accurately fixed on mine plans or the positions of surface structures shown on the mine plans in relation to underground workings may be checked before commencement of depillaring operations.

Question 13. Two vertical shafts which are about 1000 metres apart are to be connected at a predetermined datum by a straight drive of 3.0 x 2.5 m section driven from both sides. Discuss the survey controls to be exercised for the project and methods for executing the work.

Answer : SURVEY CONTROLS FOR STRAIGHT DRIVAGE

The work may be divided under the following heads :

1. A surface traverse including the two shafts to locate the correct position of shaft centres.
2. To locate the shaft centres at the bottom of the shaft.
3. To compute the co-ordinates of the shaft centres and to find out the bearing of the line joining two shaft centres.
4. An underground traverse joining shaft centres through a suitable route, if available.
5. Drivage of the roadway in the horizontal direction controlled by a theodolite.
6. To maintain the gradient of the drivage.

The surveyor should locate the exact point of shaft centres at the surface and at the bottom of the shaft by suspending plumb wires. A surface traverse and an underground traverse

through an available route should be made connecting the plumb lines. The bearing of the plumb plane at the surface should then be found out.

The horizontal directions of the drivage at the bottom of the shaft is set out with theodolite at the predetermined direction with the help of a theodolite and fixed with at least three points, solidly secured in the roof with temporary pegs spaced not more than 2-3 m apart. For setting out the direction at the shaft bottom two plumb-lines suspended from the surface at one shaft will be helpful to treat as a reference line.

Depending on the prevailing conditions the surveyor secures plumb-bobs along the centre lines of the drivage. As the drivage progresses, the driving crew checks the working for the direction. Plumb bobs are run from the fixed points and when they have come to rest the men take position behind these so that all three plumb-lines overlap. The obtained direction is projected into the face and marked with chalk. The check should be carried out at all places of drivage :

1. When pointing shotholes over the face.
2. While checking the cross-section of the working following shot-firing and loading out.
3. While setting supports.

In order to achieve high degree of accuracy the centre line of the drivage should be checked with the theodolite at an interval of progress of about 15 m.

To maintain the gradient of the drivage

A theodolite may be used. A grade line should be established on the sides of the drivage in white wash at a fixed distance usually 1 metre above the floor. Two or three pegs are inserted on the side wall at suitable intervals along the grade line. The theodolite is used to fix the position of pegs in correct gradient. A cotton string is stretched between these pegs and the line is extended and marked by chalk.

As the great importance is attached to the accuracy of the work, periodical check should be provided by levelling the drivage with a theodolite as the work proceeds.

Question 14. You are asked to determine as accurately as possible the position of a coal face in relation to the boundary towards which it is approaching, by precise magnetic methods. The coal face is bout 2000 metres from the shaft, which is 500 metres in depth, and part of the roadway leading therefrom to the face is a cross measures drift rising 1 in 3. State fully but concisely how you would carry out the work, and in particular to what parts thereof you would devote special care.

Answer : PRECISE MAGNETIC CORRELATION

The following operations are involved to determine the position of the coal face in relation to the mine boundary –

(a) A theodolite traverse from the shaft to the coal face and back again.

(b) A levelling operation to determine the difference of level between the stations so that the plan lengths of the drafts can be accurately computed.

(c) Orientation of the traverse with the surface survey.

Theodolite Traverse : The route should be reconnoitred beforehand to establish the theodolite station at the most suitable points. The stations should be so chosen as to incorporate two or more underground base lines which must be tested to prove that they are free from local attraction at both ends. The precise theodolite traverse should then be made by the double foresight method, a steel tape being used for measuring distances. A return traverse would be made to the shaft so as to close the survey polygonally and the closing error (after distribution of the permissible angular error) should not exceed 1/1500 of the total horizontal length of the traverse. Special care in centring the instrument would be necessary at the cross-measure drift as the error in the measurement of horizontal angles due to imperfect centring depends on the horizontal lengths of the drafts and not on their slope lengths.

Levelling : In the flatter section of the workings, the levelling should be carried out with a dumpy level. In the drift the theodolite should be used for reading vertical angles to compute the difference of level and plan length trigonometrically. The theodolite should be tested for horizontality of the trunnion axis to obtain the true value of the vertical angles. The index error, if any, may be eliminated by reading both vertical verniers on both faces of the instrument.

Orientation with the Surface Survey : To ensure maximum accuracy, two theodolites fitted with tubular compasses should be used for simultaneous magnetic observations on a surface reference line whose grid bearing is known and on two underground base lines previously fixed. The procedure may be outlined as follows :

1. Observe the magnetic bearing of the surface reference line at 2 minute intervals over a period of 3 hours from 11 A.M. and carefully record the results.

2. Repeat the observations on the next day but with the instruments transposed.

3. Make a similar set of observations on the third and fourth days in respect of the surface line and the second underground base.

4. Plot the results of each day's observation on graph paper and superimpose the underground curve on the corresponding surface curve so as to obtain the angle between the surface and underground bases, irrespective of any diurnal variation that may have taken place. Average the result for each underground base so as to eliminate the effect of any constant difference between the instruments and accept the average result for each base as correct.

5. Knowing the grid bearing of the surface reference line, compute the grid bearing of each of the underground bases from the known angles between the surface line and underground bases.

6. Calculate the angle between the underground bases, both from the magnetic observations and from the underground traverse. The result should agree within the permissible limits or error.

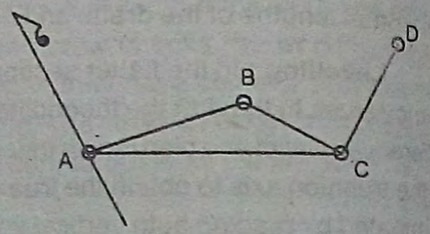
7. Calculate the grid co-ordinates of the underground survey stations so as to establish the position of the coal face accurately.

WORKED OUT EXAMPLES

Example 1. In a weisbach triangle, the azimuth of a plumb-plane marked by two wires A and B is 115°23'49", and C is a theodolite station on the south side of the eastern prolongation of AB. Given the following data, calculate the azimuth of the line CD. Illustrate your answer by a sketch.

AB = 3.481 m
BC = 2.674 m
CA = 6.155 m

Angle ACD = 179°14'33"
Angle BCD = 179010'17"



Answer : In triangle ABC :

Angle ACB = 179°14'33" – 179°10'17"
= 4'16"

As the sides of very small angles are proportional to the angles themselve, hence –

$$\text{Angle BAC} = \frac{BC \times \text{Angle ACB}}{AB} = \frac{2.674 \times 4'16"}{3.481} = 3'17"$$

Azimuth AC = 115°23'49" + 0°3'17" = 115°27'06"
Azimuth CA = 115°27'06" + 180°00'00" = 295°27'06"
Azimuth CD = Azimuth of CA + Angle ACD
= 295°27'06" + 179°14'33"
= 474°41'39" – 360° = **114°41'39"**

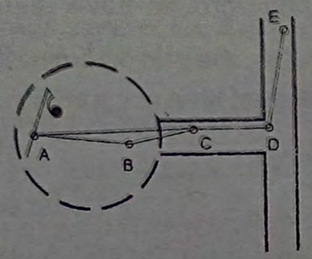
Example 2. The following are the details of observations made in connection with correlation by weisbach triangle method. A and B are the two plumb lines suspended from the pit top of the pit. D and E are the stations in the underground traverse survey which is required to be connected with the surface survey. Bearing of AB as found from the surface is 40°40'00" and the length of AB is 2.286 metres.

The observations obtained in underground are :

AB = 2.286 m
BC = 2.621 m
AC = 4.907 m
CD = 18.348 m
DE = 30.480 m

Angle BCD = 181°00'00"
Angle CDE = 96°00'00"
Weisbach angle ACB = 0°1'40"

Find the bearing of underground draft DE.



Answer : The survey is roughly plotted.

From the sine rule, we get

$$\text{Sin BAC} = \frac{BC}{AB} \text{ Sin ACB}$$

Since the angles are very small

$$\text{Angle BAC} = \frac{BC}{AB} \times \text{Angle ACB}$$

$$= \frac{2.621}{2.286} \times 100 \text{ seconds}$$

= 114.65 seconds = 1'54".

The bearing of AC = (40°40' – 1'54") = 40°38'06"
The bearing of CA = 40°38'06" + 180° = 220°38'06"
Since the angle ACB = 1'40"
∴ The bearing of CB = 220°38'06" – 1'40" = 220°36'26"
Now the angle BCD = 181°00'00", and Bearing of BC = 40°36'26"
∴ The bearing of CD = (40°36'26" + 181°) – 180° = 41°36'26"
Again the angle CDE = 96°00'00"
∴ The bearing of DE = (41°36'26" + 96°) + 180° = 317°36'26"
Hence **the bearing of the underground draft DE = 317°36'26".**

Example 3. The following are available data in weisbach triangle ABC :

Bearing of AB = S 45°26'20" W
Length of AB = 4.235 metres.
Length of BC = 5.043 metres
Length of AC = 9.280 metres
Weisbach Angle BCA = 0°12'40"
Angle ACD = 198°10'00"

D is a station on the shaft level which connects the underground survey to the Weisbach trianlge.

Calculate the bearing of CD giving neat sketch of a rectangular shaft in which the two shaft wires marked by A and B are suspended along with the weisbach triangle and the underground line CD.

Answer : The bearing of underground reference line may be calculated by the Weisbach triangle method.

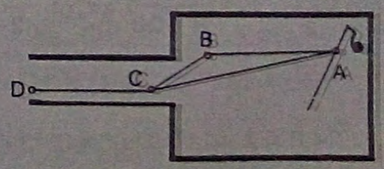
The bearing of line AB = S 45°26'20" W, the weisbach angle ACB = 0°12'40" or 760 seconds,. and the length of the plumb-plane AB = 4.235 m and BC = 5.043 metre.

From the sine rule, Sin BAC = $\frac{BC}{AB}$ Sin ACB

Since the angles are very small hence

$$\text{Angle BAC} = \frac{BC}{AB} \times \text{Angle ACB (in seconds)}$$

$$= \frac{5.043}{4.235} \times 760" = 905" = 15'05"$$



The bearing of BC = S (45°26'20" – 0°15'05") W

= S 45°11'15" W = 225°11'15" Azimuth.

Now, the angle ACD = 198°10'00"

Hence the bearing of CD

= (225°11'15" + 198°10'00") – 180°

= 243°21'15" = S 63°21'15" W

The bearing of the underground reference line CD = S 63°21'15" W.

Example 4. In weiss quadrilateral angles are measured at two points A and B with the object of determining the azimuth of PO.

Azimuth of AB = 89°42'

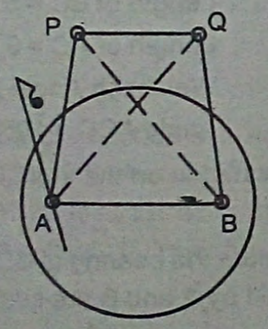
Angle PAQ = 39°54' Angle QAB = 42°19'

Angle PBQ = 41°08' Angle ABP = 44°24'

Length of AB = 30 meters.

Answer : PROBLEM ON WEISS QUADRILATERAL

In the given data, the length of AB = 30 m, and the bearing of AB = 89°42'. The angle PAQ, BAQ, PBQ and ABP are also given.



Angle ABQ = (44°24' + 41°08') = 85°32'

Angle PAB = (39°54' + 42°19') = 82°13'

Angle AQB = 180° – (85°32' + 42°19') = 52°09'

Angle APB = 180° – (82°13' + 44°24') = 53°23'

Now from the sine rule in the triangle ABQ :

$$BQ = \frac{\text{Sin QAB}}{\text{Sin AQB}} \times AB = \frac{\text{Sin } 42°19'}{\text{Sin } 52°09'} \times 30 = 25.5779 \text{ metres.}$$

From the triangle APB :

$$PB = \frac{\text{Sin PAB}}{\text{Sin APB}} \times AB = \frac{\text{Sin } 82°13'}{\text{Sin } 53°23'} \times 30 = 37.0321 \text{ metres.}$$

In the triangle PQB :

$$\tan\frac{Q-P}{2} = \frac{PB-QB}{PB+QB} \times \text{Cot}\ \frac{B}{2} = \frac{37.0321 - 25.5779}{37.0321 + 25.5779} \times \text{Cot}\ \frac{41°08'}{2} = 0.2094$$

$$\therefore \frac{Q-P}{2} = \tan^{-1}(0.2094) = 11°49'50"$$

$$\therefore Q - P = 2 \times 11°49'50" = 23°39'40"$$

Again Q + P = 180° – 41°08' = 138°52'

Hence $\angle Q = \frac{1}{2}$ (22°39'40" + 138°52') = 81°15'50"

$\therefore \angle BQP = 81°15'50"$

Given the bearing of AB = 89°42', Angle ABQ = 85°32'

$\therefore$ The bearing of BQ = (89"42' + 85°32') + 180° = 355°14'

But the angle BQP = 81°15'50"

$\therefore$ The bearing of QP = (355°14" + 81°15'50") – 180° = 256°29'50"

$\therefore$ The bearing of PQ = (256°29'50" – 180°) = 76°29'50"

Hence **the bearing of the reference line PQ = 76°29'50"**

Example 5. Two plumb lines E and W are suspended in a vertical shaft. Bearing and distance of EW and co-ordinates of E as determined from the surface survey are :

Azimuth of EW = 265°33'; Distance of EW = 1.3807 m

Co-ordinates of E = + 542.241 m and + 280.112 m

A theodolite is set up at T south of plane EW in a cross cut going east and the following observations are made :

$\angle$WTE = 0°02'20" $\angle$WT (205) = 183°20'30"

Distance of TE = 7.6252 m; TW = 12.1555 m and T (205) = 40.2014 m.

Then the theodolite is set up at the station (205) and the following measurements are obtained :

$\angle$T (205) (206) = 178°25'00" and Distance (205) (206) = 52.2055 m.

Find the bearing of underground base line (205) (206) and the co-ordiantes of station (206).

Answer : In the weisbach triangle EWT

The weisbach angle ETW = 2'20" = 140 seconds.

$$\angle EWT = \frac{ET}{EW} \text{ Angle ETW} = \frac{7.6252}{1.3807} \times 140" = 12'53"$$

Since the bearing of EW = 265°33'

$\therefore$ The bearing of WT = (262°33' – 180°)

+ 12'53" = 85°45'53"

The bearing of TE = (85°45'53" – 180°)

+ 2'20" = 265°48'13"

The angle WT (205) = 183°20'30"

$\therefore$ The balance of T (205) = [(265°48'13" – 180°) + 183°20'30"] – 180° = 89°06'23"

Again the angle T (205) (206) = 178°25'00"

$\therefore$ The bearing of (205) (206) = (89°06'23" + 178°25'00") – 180° = 87°31'23"

The co-ordinates of respective points are calculated and tabulated as under :

Line	Distance	Azimuth	Partial Co-ordinates		Stn	Total Co-ordinates	
			Lat	Dep		Lat	Dep
					E	+ 542.2410	+ 280.1220
ET	7.6252 m	85°48'13"	+ 0.5579	+ 7.6047	T	+ 542.7985	+ 287.7267
T (205)	40.2015m	89°06'23"	+ 0.6269	+40.1966	(205)	+543.4258	+337.9233
(205)(206)	52.2055m	87°31'23"	+2.2561	+52.1567	(206)	+545.6819	+380.0800

Hence the bearing of the underground reference line (205) (206) = 87°31'23" and the co-ordinates of the station (206) = + 545.6819 m and + 380.0800 m.

Example 6. A and B are plumb lines suspended in a shaft. A staple Z is to be sunk from an upper to a lower level in the workings and the following are notes of surveys to determine the position at Z.

	Line	*Bearing*	*Distance (m)*
Upper Level	A – B	Due North	3.0
	B – C	Due North	13.5
	C – D	N 60° E	12.0
Lower Level	B – A	Due South	3.0
	A – W	Due South	15.0
	W – X	S 5°W	39.0
	X – Y	N 80°E	37.6
	Y – Z	N 3°W	–

Calculate the distance from D on the line CD extended to the point exactly above the line YZ.

Answer : The co-ordinates of the two surveys are calculated and the partial and total co-ordinates of the stations are tabulated, using station A as the point of origin, as under –

Line	Bearing	Dist	Partial Co-ordinates		Total Co-ordinates		Stn
			Lat	Dep	Lat	Dep	
					0	0	A
AB	Due North	3.0	+ 3.0	0	+ 3.0	0	B
BC	Due North	13.5	+13.5	0	+16.5	0	C
CD	N 60° E	12.0	+ 6.0	+ 10.39	+ 22.5	+ 10.39	D
AW	Due South	15.0	– 15.0	0	– 15.0	0	W
WX	S 5°W	39.0	– 38.85	– 3.40	– 53.85	– 8.40	X
XY	N 80°E	37.6	+ 6.53	+37.03	–47.32	+33.63	

Difference in Departure Y and D = 33.63 – 10.39 = 23.24 m

Difference in Latitude Y and D = 47.32 + 22.50 = 69.82 m

$$\text{Tan bearing YD} = \frac{\text{Departure}}{\text{Latitude}} = \frac{23.24}{69.82} = 0.332856$$

Therefore bearing YD = N 18°24'37" W

Length YD = 69.82 Secant 18°24'37" = 73.586 m

Referring to triangle DYZ –

Angle DYZ = Bearing YD – Bearing YZ

= N 18°24'37" – N 3°00'W = 15°24'37"

Angle XZD = Bearing YZ + Bearing CD

= N 3°00' W + N 60°00' E = 63°00'

Applying the sine rule, we have –

$$\frac{DZ}{\text{Sin DYZ}} = \frac{YD}{\text{Sin YZD}} \quad \therefore \; DZ = \frac{73.586 \text{ Sin } 15°24'37"}{\text{Sin } 63°} = 21.946 \text{ m}$$

Therefore, the line CD will have to be extended a further 21.946 metres to point Z. it is proposed to sink the staple pit.

Example 7. In making a correlation of underground and surface surveys at a mine by means of plumb lines A and E hanging in separate shafts, the following data were obtained from an underground traverse made with the theodolite :

Line	*Assumed Azimuth*	*Distance*
AB	150°	303 m
BC	90°	150 m
CD	44°	363.5 m
DE	300°	63.15 m

If the true azimuth of the line AE at the surface is 333°31', calculate the true azimuths of the lines in the underground traverse as accurately as possible.

Answer : The following table shows the calculated co-ordinates of the various points in the traverse, from which the bearing and distance of the line connecting the two shafts are obtained.

Line	Azimuth	Q. B.	Distance in mts. L	Latitude L Cosθ		Departure L Sinθ		Stn.
				+	–	+	–	
AB	150°	S 30°E	303					
					262.40	151.50	–	B
BC	90°	Due East	150					
				–	–	150.00	–	C
CD	45°	N 45°E	363.5					
				257.03	–	257.03	–	D
DE	300°	N 60°W	63.15					
				31.575	–		54.689	E
		TOTAL		+ 288.605	– 262.40	+ 558.53	+ 54.689	
				– 262.400		– 54.689		
		Total Co-ordinates of E =		+ 26.205		+ 503.841		

Natural tangent of bearing line AE = $\frac{03.841}{26.205}$ = 19.227

∴ Bearing of line AE N 87°1'E

and Azimuth of Line AE = 87°01'

Difference between surface and underground line connecting the two shafts

= (360° – 333°31') + 87°01' = 113°30'

True azimuth of line AB = 150° – 113°30' = 36°30'

True azimuth of line BC = 90° – 113°30' + 360° = 336°30'

True azimuth of line CD = 45° – 113°30' + 360° = 291°30'

True azimuth of line DE = 300° – 113°30' = 186°30'.

Example 8. The following are the notes of an underground traverse from an assumed base between plumb wires marking the centres of two shafts A and B.

From Shaft A	Azimuth	Length
	270°	73.5 m level
	181°	116.7 m level
	176°	516.6 m dipping 9°
	171°	120.0 m level
	90°	132.0 m level to shaft B

The bearing of the line BA on the surface is N 6°30'W. Calculate the azimuth of the underground traverse lines relative to surface base.

Answer : Line number 3 whose azimuth is 176° has an inclination of 9°, hence the horizontal length of the line = 516.6 m x Cos 9° = 505.235 m.

Taking the co-ordinates of the centre of the shaft A as zero, the false co-ordinates of all the stations are calculated.

Line	Azimuth	Q. B.	Hor. Distance	Local Co-ordinates		Total Co-ordinates		Stn.
				Lat.	Dep.	Lat.	Dep.	
1.	270°	Due West	73.5			0	0	A
2.	181°	S 1° W	116.7		+ 73.5	0	– 73.5	
3.	176°	S 4° E	526.235	– 116.68	– 2.0366	– 116.68	– 75.5366	
4.	171°	S 9° E	120	– 504.00	+ 35.242	– 620.68	– 40.2946	
5.	90°	Due East	132	– 118.52	+ 18.772	– 739.20	– 21.5226	
					+ 132.00	– 739.20	+ 110.4774	B

False bearing of BA = $\tan^{-1} \frac{110.3774}{739.20}$ NW = $\tan^{-1}$ 0.14945

= N 8°30'W; Azimuth = 351°30'

Correct bearing of BA = N 6°30'W (Given) Azimuth of BA = 253°30'

Difference between false azimuth and correct azimuth of BA = 2°

(less than the correct value)

Hence the correct azimuth of the underground traverse lines –

Line	Flase Azimuth	Correlation	Correct Azimuth
1	270°	+ 2°	272°
2	181°	+ 2°	183°
3	176°	+ 2°	178°
4	171°	+ 2°	173°
5	90°	+ 2°	92°

Example 9. A colliery plan has been laid down on the National Grid of the Ordnance Survey and the co-ordinates of two stations A and B in metres and reduced to A as local origin are :

Station	A	O	O
Station	B	32.76E	297.51S

Calculate the grid bearing of the line AB. The mean magnetic bearing of the line AB S 3°54'40"W, and the mean magnetic bearing of an underground line CD is N 17°54'20"W. State the grid bearing of the line CD.

Answer : Grid bearing of AB = $\tan^{-1} \frac{32.76}{287.41}$ = S 06°17'E

Mean Magnetic bearing AB = S 03°54'40"W

∴ Declination = 03°54'40" + 06°17' = 10°11'40"

Magnetic bearing of CD = N 17°54'20"W

∴ Grid bearing of CD = 17°54'20" + 10°11'40" = N 28°06'W.

Example 10. The following are the notes of a traverse made underground between wires suspended in two shafts of a colliery.

The wires have been connected by a surface survey and the bearing of the line joining wires 1 and 2 related to the Grid meridian is N 58°15'E. Assuming an arbitrary bearing of due north for line 1–A, calculate the bearings of the underground traverse between the wires. Determine the Grid Bearing of the underground base line AB.

Line	Length in metres	Clockwise angles
1 – A	29.10	
A – B	45.96	Angle 1AB = 227°15'
B – C	30.60	Angle ABC = 210°10'
C – 2	45.60	Angle BC2 = 269°18'

Answer : In following table shows the calculated co-ordiantes of various underground station points.

Line	Quadrant Bearing (θ)	Distance Metres L	Latitude L Cos θ	Departure L Sin θ
1 – A	Due N	29.10		
A			+ 29.100	–
A – B	N 47°15'E	45.96		
B			+ 31.200	+ 33.750
B – C	N 77°15'E	30.60		
C			+ 6.753	+ 29.845
C – 2	S 13°17'E	45.60		
2			– 44.380	+ 10.477

Total co-ordinates of station 2 = N 22.673, E 74.072

$\tan \theta = \frac{74.072}{22.673}$ $\therefore \theta = 72°59'$

Traverse bearing of 1 – 2 = N 72°59'E

Grid bearing of 1 – 2 = N 58°15'E

∴ Difference = 14°44'

Traverse bearing of AB = 227°15' – 180° = N 47°15'E

∴ **Grid bearing of AB** = N (47°15' – 14°44')E = **N 32°31'E**

Example 11. Underground and surface surveys at a colliery are to be connected by reference to single wires suspended in shafts 1 and 2 respectively. Azimuth of the line between wires 1 and 2 obtained from surface survey is 202°14'20".

Undernoted are clockwise angles and horizontal distances of an underground survey made from 1 to 2

Angle		Horizontal distances
1 AC	91°09'30"	1A = 45 m
ACD	192°35'00"	AC = 61.8 m
CDE	165°27'40"	CD = 55 m
DE2	88°43'10"	DE = 38.8 m
		E2 = 46.4 m

During the survey an underground baseline AB was set out and the value of angle 1AB was found out to be 178°29'20". Calculate the azimuth of the line AB in relation to that between the shaft wires at the surface.

Answer : CORRELATION PROBLEM

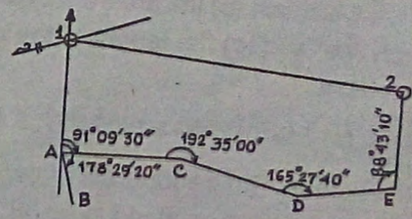
Assuming the line A1 of underground traverse is the meridian, the co-ordinates of the underground traverse stations are calculated as follows :

Line	Angle	Azimuth θ	Distance L mts	Latitude L x Cosθ	Departure L x Sinθ
1 A		180°	45		
A				– 45	
AC	1AC = 91°09'30"	91°09'30"	61.8		
C				– 1.2493	+ 61.7874
CD	ACD = 192°35'00"	103°44'30"	55		
D				– 13.0649	+ 53.4257
DE	CDE = 165°27'40"	89°12'10"	38.8		
E				+ 0.5398	+ 38.7962
E2	DE2 = 88°43'10"	357°55'20"	46.4		
2				+ 46.3695	– 1.6823
Total Co-ordinates of Stn. 2 =				– 12.4049	+ 152.327

Traverse bearing of line 1–2 = $\tan^{-1} \frac{152.327}{-12.4049}$

= S 85°20'39"E.

or, Azimuth 94°39'21".



Actual bearing of line 1–2 = 202°14'20"

Difference = 202°14'20" – 94°39'21" = 107°34'59".

Hence **Azimuth of the line AB** in relation to shaft wires 1–2 at surface = 178°29'20" + 107°34'59" = **286°04'19".**

PROBLEMS

Problem 1. B and C are two plumb lines hung in a vertical shaft. The azimuth of BC is 353°45'30", and the distance between the wires is 2.145 m. A theodolite is set up at a point A, situated to the east of the southern prolongation of BC; the angle BAC is found to measure 0°1'50", AB measures 2.262 m and AC 4.407 m. Find the azimuth of the line CA. **[Ans. 173°43'34"]**

Problem 2. The correlation of the surface and underground surveys of a colliery was based on two wires, A and B, No. 1 shaft, the surface bearing of which was 116°02'.

Later, it was possible to check the correlation by a traverse from A to a wire C in No. 2 shaft. The underground traverse from A to C base on AB was as under :

From wire A :	Line 1	Azimuth 116°02'	22.26 m
	Line 2	Azimuth 92°24'	130.01 m
	Line 3	Azimuth 35°01'	467.56 m

Line 4	Azimuth 83°12'	252.59 m To Wire C.

The surface bearing AC was found to be 59°15'. What was the error in the underground bearing of AB ? **[Ans. 0°01'30" too great]**

Problem 3. In making a correlation of the underground and surface surveys at a mine, by means of plumb-lines. A and E hanging in separate shafts, the following data were obtained :

The azimuth of line AE at the surface = 181°25'. Underground traverse made with the theodolite reading clockwise –

Line	Distance	Assumed Azimuth	Vernier Angles
AB	57.2 m	0°00'	
BC	196.7		179°22'
CD	61.6		184°17'
DE	29.6		91°27'

Calculate (a) the arbitrary bearing of AE in the underground survey; (b) the true azimuth of each line; and (c) the length of the closing line AE, from both the arbitrary and the correct co-ordinates.

[Ans. (a) 355°01'; (b) S 06°24'W, S 05°46'W, S 10°03'W, S 78°36'E, (c) 319.16 m and 319.14 m]

Problem 4. The bearing AB of two plumb-wires A and B hanging in a vertical shaft has been determined by co-planing to be 89°15'25". At the shaft bottom a station C has been fixed to the east of B and the following measurements made : CB 2.595 m; AB 3.81 m; CA 6.402 m, angle ABC 00°02'08". The angle BCD is 169°52'12". D being a traverse station. Determine the bearing of the underground traverse line CD. **[Ans. 79°11'12.2"]**

Problem 5. The following observations were obtained in making a correlation survey to two shaft wires J and H :

Line	Bearing	Distance
FG	N 84°33'30"E	47.1381 m
GJ	S 2°51'00"E	89.2531 m
FH	S 5°53'00"E	90.0287 m

Calculate the bearing and distance of the line joining the wires J and H.

[Ans. 42.4185 m; S 83°21'59"W]

CHAPTER V

PHOTOGRAMMETRY

PHOTOGRAMMETRY OR PHOTOGRAPHIC SURVEYING

Photogrammetry or photommetry or photographic surveying is the science and art of obtaining measurements accurately by use of photographs for various purposes such as the construction of planimatric and topographic maps, classification of soils, interpretation of geology, acquisition on military intelligence and preparation of composite pictures of the ground. The photographs are taken either from the air or from station on the ground. Photogrammetry may be divided into two classes :

1. Terrestrial or ground photogrammetry,
2. Aerial photogrammetry or aerial surveying.

Terrestrial Peotogrammetry : The system of surveying in which plans or maps are prepared from photographs that are taken from a ground station, the position of which is usually known or can be readily determined. It is suitable for small-scale mapping of open hilly or mountaineous countries. It is well adapted to topographical or preliminary surveys. It is not suitable for flat or wooden country.

Aerial Photogrammetry : The system of surveying in which plans or maps are prepared from photographs taken by a camera mounted in an aircraft flying over the area. Aerial surveying is used with great success for reconnaissance and preliminery surveys of buildings, towns, etc. It is particularly suitable for inaccessible regions, hidden areas, unhealthy regions or wooded country.

TERRESTRIAL PHOTOGRAMMETRY

The principle of terrrestrial photogrammetry is exactly similar to that of plane table surveying i.e. if the directions of same objects photographed from two extremities of measured base are known, their positions can be located by the intersection of the two rays to the same object. The difference is that most of the work is executed in the field by plane tabling, while in photogrammetry it is done in the office.

The terrestrial photogrammetry can be divided into two branches :

(a) Plane table photogrammetry.

(b) Terrestrial stereophotogrammetry.

The *plane table photogrammetry* consists essentially in taking photograph of the area to be mapped from each of the two or three stations. The photograph perpendiculars may be oriented at any angle to the base, but usually from an acute angle with the latter. The main difficulty arises in the identifications of image points in a pair of photographs. In the case of homogeneous areas of sand or grass, identification becomes impossible. The principles of stereophotogrammetry, however, produced the remedy.

In *terrestrial stereophotogrammetry*, due to considerable improvement of accuracy obtained by the stereoscopic measurement of pairs of photographs, the camera base and the angles of intersection of the datum rays to the points to be measured can be considerably reduced since the camera axes at the two stations exhibit great similarity to each other. The image points which are parallactically displaced relative to each other in the two photographs are fused to single spatial image by the stereoscopic measurement.

PHOTO-THEODOLITE

The photo-theodolite is a combination of 1-second theodolite and a terrestrial camera, and is used for taking photographs and measuring the angles which the vertical plane of collimation makes with the base line.

A photo-theodolite essentially consists of the following parts :

1. A *camera box of fixed focus type*. The focal length of the lens is generally 15 cm or more. The camera box is mounted on the axis in the same manner as the vernier plate of a theodolite. Thus the box can be rotated in azimuth about its vertical axis.

2. A *hollow rectangular frame* placed vertically to the rear side. The frame carries two cross-hairs, the intersection of which is exactly opposite to the optical centre to the lens. The cross-hairs are pressed tightly against sensitive plate and are thus photographed on the photographic plate along with the field object. Two small celluloid strips can be fitted into the grooves in the lower corners of the frame and can be easily removed to write any description upon them in ink which is also photographed.

3. A *horizontal transparent celluloid tangent scale* attached to the frame across its rear side.

4. A *circular magnetic compass* mounted on the base of the frame. The pivoted magnetic needle carries a vertical cylindrical transparent scale graduated to 30 minutes. The magnetic reading of the vertical plane is given by the reading of the scale at its intersection with the vertical hair on the photograph.

5. A *sensitive photographic* plate is placed between the vertical frame and the back which is held by the spring. Before uncapping the lens, the front of the side is withdrawn to expose the plate and the vertical frame is moved backward and forward by a screw until the tangent scale and the hair lens are in contact with the plate. The magnetic needle is also set free to swing on its pivot. When the lens is uncapped (after the needle comes to rest), the photographs of hairlines, tangent scale, and the circular scale are imprinted on the negative. The reading of the scale as

its intersection with the vertical hair on the photograph gives the magnetic bearing of the principal vertical plane (i.e. the vertical plane containing the optical axis).

6. A telescope is mounted on the top of the camera box which can be rotated in a vertical plane, about a horizontal axis. It is fitted with a vertical arc with verniers, clamp and slow-motion screw. The line of sight of the telescope is set in the same vertical plane as the optical axis of the camera lens.

7. The camera box is supported on the tripod and is furnished with an inner and outer axis, each of which is fitted with a clamp and fine adjusting screw. A graduated horizontal circle carries verniers reading to single minutes and fitted with clamp and tangent screw below the camera. One or two sensitive bubble are mounted on the top of the camera box for levelling purposes.

8. The camera box along with all its component parts is supported on a levelling head carrying three foot screws.

Example : Phototheodolite by M/S. Heerbrugg Ltd.

AERIAL PHOTOGRAMMETRY

Aerial photogrammetry applies to the measurement of photographs taken from the air. It is mostly suitable for small-scale mapping especially in flat country. The advantages of this method are –

1. It is suitable for inaccessible regions and hidden areas or wooded country.
2. It can be successfully used for reconnaissance and preliminary surveys for roads, railways, etc.
3. It provides informations required for planning of land drainage, irrigation and flood control.
4. Land reclamation schemes can be planned from air photographs.
5. Volumes of stock piles of materials, of excavation of quarries may be obtained from stereopairs and results are made available almost at once.
6. Earthwork can be computed using a computer linked with the stereo plotting machines.
7. Selection and survey of catchment areas, and reservoir sites are facilitated by the study of stereopairs. Capacity of reservoirs can be worked out within shortest time.
8. Surveys for assessing damage due to earthquakes, subsidence in mining areas can be estimated from photographs.

Aerial survey should be carried out with great skill, hence skilled, specialised and experienced personnels are required for its performance. Being expensive and very elaborate it is generally carried out by government agencies such as Survey of India Department in India. The survey is carried in four operations–

1. Flying
2. Aerial photography
3. Ground control
4. Mapping.

The equipment required for aerial survey are –

(a) An aeroplane

(b) An aerial camera

(c) Photographic materials for development

(d) Accessories required for interpretation and plotting.

1 **Flying :** To keep the scale of the photograph same the speed and height of the aeroplene should be uniform. The changes in height will change the scale of the photograph and tilting of camera due to this will cause distortion in the photograph.

2. **Aerial Photography :** Air photographs are taken with special types of cameras fitted in the aeroplane automatically. The photographs are taken with sufficient overlap both in the direction of flight (About 60% overlap) and at right angles to it (About 25% laterally).

The following are three ways of taking aerial photographs.

(a) Vertical Photographs

(b) Oblique Photographs

(c) Composite Photographs

(a) *Vertical Photograph* : When the axis of camera at the time of the exposure is pointing verticaly downward, and the position of the flying aircraft is horizontal, an exposure made in this position results in a vertical photograph. Vertical photography becomes uneconomical if the scale is much less than 1 / 30,000.

Vertical photograph is useful for large scale mapping of great precision. It gives accurate results and is therefore preferred

(b) *Oblique Photographs :* When the axis of the camera at the time of exposure is inclined, the photographs taken are called oblique.

The tilt of the camera ranges from 20 to 30 degrees in case of low oblique photographs and 30 to 90 degrees in case of high oblique photographs.

Oblique photographs are more pictorial than vertical and therefore are easily interpreted. Further oblique photographs are used for small scale mapping and when a contoured map of the area is required.

(c) *Composite Photographs* : When vertical photographs and oblique photographs are taken simultaneously by two camera mounted in the air craft photographs taken are called composite photographs.

By this method more area is covered.

3. **Ground Control :** It is equally important to produce accurate maps from aerial photographs. The greater is the accuracy of the resulting map if there are large number of ground control points. The ground control is done by two methods :

(i) Pre-pointing method

(ii) Post-pointing method.

(i) *Pre-pointing Method :* In this method, the ground control is established before photographs are taken. For the purpose some easily well defined natural objects are fixed as ground stations. The object being that these ground control stations should be easily recognised by the photographer while flying in air plane and be also identified on the photograph. If the natural objects are not easily obtained, some other artificially marks are fixed on the ground by white wash, paint or by any other means of sufficient size so as to be identified from high altitudes.

(ii) *Post-pointing Method :* In this method the ground control points are fixed on photographs after photography, correctly after careful verification by a ground surveyor on the field. A slight error in identification may affect the accuracy of the resulting map. The control points on photographs should be sharp and clear such as the junction of roads, bank corners, railway crossings and other distinct points.

The ground surveyor and air surveyor should be familier in aerial surveying and mapping to achieve good and accurate results.

The points are established by triangulation or traversing

4. **Mapping** : Plotting of maps from photographs taken from air can be done by the following methods–

(i) Radial line method.

(ii) Slotted template method.

(iii) The three-point method.

The methods are costlier as they are slow and tedious.

A stereoscopic plotting instrument, or plotter, is used to plot planimetric and topographic maps to a predetermined scale by using photographs that have been taken with a precision aerial camera. The elevations of selected ground points measured in a plotter can be determined with varying degrees of accuracy, the error depending on the precision of the plotter, this ranges from 1 / 1000 times the flying height to as little as 1 / 10,000 the flying height. The error in contour lines drawn by means of the plotter will vary from 1 / 500 to 1 / 2500 times the flying height. In order to maintain accuracy, the photographs used in a plotter are printed on glass plates.

Air survey plotting machines or instruments are many and varied. They have been devised for direct plotting from stereo pairs. The principles on which the machines work are as follows :

In the machines photographs can be set and adjusted in such a way that the effects of height, tilt are removed and a stereoscopic image of the ground showing relief is obtained. A floating mark provided in the machine can be brought in contact with any point on the surface of the stereoscopic image model. The floating mark is connected to the tracing point and as the mark moves, tracing point also moves on the drawing paper producing a map of the ground. Also by setting a floating point to a point of known elevation, contours can also be traced on the pla

Examples of different classes of plotting machines are : Multiplex (Zeiss), Balplex plotte Kelsh plotter are all double projector plotters; wild Autograph A_7, is a universal first order instrume used for aero triangulation and for detail plotting.

AERIAL CAMERA

Aerial photographs are taken by a camera using a large format size film which is automatically moved after each exposure which is also made automatically at regular time intervals. The duration of the intervals is governed by the conditions of the survey. The *Aerial Camera* is comparable to a surveying instrument in that it gathers data in the form of light rays with certain directions and records the data on a photographic negative. The primary function of this camera is taking pictures. But since the aerial camera is mounted on a fast moving aeroplane it requires :

(i) fast lens, (ii) high speed and efficient shutter, (iii) high speed emulsion for the film, and (iv) a magazine to hold large rolls of film.

An aerial camera consists of the following essential parts :

1. Lens assembly.
2. Camera cone.
3. Focal plane.
4. Camera body.
5. Drive mechanism.
6. Magazine.

The lens assembly, which includes the shoutter and diaphragm, forms the photographic image with the proper amount of light admitted. The lens itself is composed of several elements. It must be highly corrected to satisfy the requirements of a rather wider angular coverage, a high resolution, and a very minor amount of lens distortion. Because of atmospheric haze which contains an overabundance of blue light, the cone and the camera body hold the lens assembly in its proper position with respect to the focal plane.

The camera body houses the drive mechanism for the shutter assembly and the magazine. The upper surface of the camera body defines the focal plane of the camera. This surface contains four fiducial or collimation marks, located either with one in the middle of each side of the focal plane opening or else in each corner. The shutter speed generally varies from 1 / 100 second to 1/1000 second.

The drive mechanism is housed in the camera body and is used for (i) winding and tripping the shutter, (ii) operating the vacuum system for flattening the film, and (iii) winding the film. It may be operated manually or automatically.

The magazine is a light container for the film. It contains a feed spool and a take-up spool, the mechanism for advancing the film after each exposure, and a device for holding the film flat in the focal plane at the instant the exposure is made.

The two points on the axis are the front and rear nodal points of the lens system. On striking the front nodal point any ray of light is so refracted by the lens that it appears to emerge from the rear nodal point parallel with its original direction.

The camera is suspended in a camera mount by means of a pair of trunions on either side

of the camera. It is free to rotate in the mount about all three axes. The mount is secured over an opening in the bottom of the airplane.

Example : Wild RC–9 automatic super wide angle camera ; Zeiss aerial survey camera.

Single Lens Camera : The single lens camera is very simple and robust in construction and free from optical defects. It is generally equipped to take roll films, and can accomodate 200 frames or more. The range of focal length is normally 15 cms and 60 cms. The common apertures are f/4 to f/7 but in certain Zeiss wide angle type, the aperture employed is f/6.3 or as in the Ross ultra wide type, f/5.5. The shutter is designed to eliminate the distortion of image on the photograph. Shutter speed varies from 1/50 to 1/300 sec, while the sizes of photographs are usually 18 cms x 18 cms or 23 cms x 15 cms. or 20 cms. x 20 cms.

Wild RC–9 camera, with ultra wide angle super Aviogon lens, with 88 mm focal length, gives 120^0 coverage along the diagonals of a 23 cms. x 23 cms. photo format.

Multi–lens Camera : These are used for taking vertical photographs as well as oblique photographs. The camera is made of several independent chambers, each equipped with a separate lens, shutter and film magazine, all of which are synchronously operated. There are various models. In the Zeiss, 4–lens type, there are four oblique chambers. In the Fairchild, 5–lens type, one central vertical chamber surrounded by four oblique chambers. In the Barr and Strond 7–lens type there is one central vertical section and 6 oblique sections.

There are also slight differences in the mode of attachment of the camera to the cock-pit of the aircraft, but whatever the type of fixture, the purpose is to compensate by differential motion any accidental tilt of the aircraft, during the flight and for maintaining horizontality while taking vertical photos.

Panchromatic film is invariably used. distortion proof nitrate film base is employed, but this is highly inflammable. However, distortion proof nitrate safety film is also available. Material having an uniform shrinkage of only 0.015 per cent is now available.

Accessories and Auxiliary Equipment : (i) Spirit level, (2) View finder, (3) Intervalometer– a device which enables the taking of photographs automatically at calculated intervals of time, (4) Colour filters with various film emulsions for bringing out specific features; (5) Altimeter for recording heights and in the improved cameras statoscopes are attached.

In practice, provision is also made in the aircraft to accommodate geophysical equipment, for airomagnetic surveys or radiometric surveys, and also an air borne profile recorder.

The types of Aircraft employed are D C. 3–Dakota, D.H. Dove Canse, Airocommander, Mosquito and Survey Prince. The normal flying altitudes are of the order 600 m and occasionally 7,500 m, while the photo scales, of cameras with lens of 15 cms focal length, vary between 1 : 40,000 and 1 : 50,000.

QUESTIONS & ANSWERS

Question 1. What is photographic survey ? What are its advantages ? Describe the field work of terrestrial photographic surveying.

Answer : PHOTOGRAPHIC SURVEY : ADVANTAGES & FIELDWORK

Photographic Survey : Photographic surveying or photogrammetry or photommetry is the science of making measurements on photographs. It relies on photographs taken from stations on the ground. Since photographs do not provide exact plans of the terrain to be surveyed, they must be processed in accordance with the objects surveyed and their images. The discipline dealing with the methods of processing photographs to prepare precise plans of the field and locate the objects being surveyed spatially is photogrammetry.

Advantages : The important advantages of photographic surveys are as follows–

1. When the surveying is to be carried out in difficult areas having great undulations as in the case of hilly areas, where the use of ordinary surveying instruments become difficult, photographic surveying is best suited.

2. By this type of surveying the inaccessible regions and hidden areas or wooded country can be located easily by taking photographs.

3. The flooded area can also be easily located by this type of surveying.

4. As the science of aerial photogrammetry has developed, it has come to include all operations, products involving the use of aerial photographs. Among thess are included the measurement of horizontal distances, the determination of elevations, the compliation of planimetric and topographic maps, in preparation of mosaics and orthophotos, and the interpretation and analysis of aerial photographs for geological, mining and other engineering investigation and evaluation.

Field Work of Terrestrial Photommetry : The field work of the surveying consists of (1) Reconnaissance, (2) Triangulation, and (3) Camere work.

Reconnaissance : A careful study of the available maps of the area to be surveyed should be made to select most suitable camea stations. The stations should be so selected that the area would be covered with a minimum number of photographs and the work can be done with speed and economy.

The area should next be reconnoitered with a view to selecting suitable triangulation stations and camera stations.

The following points are to be considered in selecting camera stations :

(a) The stations should be so fixed that the objects to be plotted on the map can be clearly and easily recognised on at least two photographs taken at different stations.

(b) The angle of intersection of the two direction lines locating a particular point should not be too acute or too obtuse.

(c) The direction of the pointing should as far as possible be normal to the slope of the ground.

(d) They should be fixed on higher points so as to command the area.

Triangulation : All camera stations should be connected by a triangulation system. The triangulation stations may sometimes be used as camera stations. The elevations of camera stations should be determined by direct or by trigonometrical levelling.

Camera Work : Photographs are taken in pairs from the ends of a base line (i.e. a line joining the camera stations) which is carefully measured. The more important points should appear on three or more photographs, and each photograph should contain at least one triangulation station or a point which has been already fixed from a camera statiion. Adjacent photographs should sufficiently overlap. The number of photographs to be taken at each station depends upon the area to be mapped, and the field or view of the camera.

Question 2. Describe the ground control for photogrammetry. What are the reasons for keeping overlap in the photograph ?

Answer : GROUND CONTROL AND OVERLAP IN PHOTOGRAPHS

Ground Control : In order that aerial photographs may be used for making simple measurements of distances and elevations a certain amount of ground control is necessary to fix the scale of the photographs, and to establish a vertical datum with contour lines on the map. This control may be obtained simply by measuring the ground distance between two identifiable points that appear on the photograph. An elaborate system of triangulation and level nets may be required to control the horizontal and vertical positions of identifiable points that appear on the photographs. In general, triangulation and traverse stations and bench marks cannot be identified on the lphotographs unless they have been marked in the field before the pictures were taken. As a result shorter traverse lines and level lines, usually of lower order of accuracy than the syustem, are run to points that can be identified. These points are called picture points, and the network of picture points in a group of photographs is called photo control.

The horizontal photo control may be obtained by running a transit-tape, transit-stadia, or plane-table traverse from a fixed point to the picture points, or it may be obtained by a single triangulation system of the desired order of accuracy. A large amount of photo control work is conducted by a combination of theodolite and an electronic distance measuring instrument. Vertical photo control is obtained by running differential, stadia, or plane-table levels from bench marks to the points, or, for large controur-intergral mapping by using aneroid barometers.

The choice of the field method to be used in establishing photo control depends on the desired accuracy, which in turn depends on the scale of the map or mosaic and the contour interval to be used in compiling the map.

A picture point must be positively identifiable between the photograph and the ground. It must be sharp and well-defined as seen on the photograph under magnification. A picture point must fall in the correct position on the photograph. Such point falling near the very edge of the photo may be useless. It should be reasonably accessible on the ground so that the expense of the photo-control survey can be kept to a minimum. Finally, the picture point must be well described and documented directly on the photograph used in the field when photo-control survey is performed.

When a high degree of accuracy is required in the photogrammetric process, natural features are generally not used as picture points. Instead, the field survey crew sets out panels, known as premarked panels or panel points, just prior to photography.

Reasons for Overlap : When aerial photography is used for mapping, mosaic construction, or orthophotography, flights are laid out on a flight map with a spacing that will cause photographs to cover an overlapping strip of ground. In order to facilitate mapping from vertical photographs, an overlap of about 60% is maintained between successive photographs in the direction of flight (forward overlap or longitudinal overlap) and an overlap of about 30% is allowed between adjacent strips (lateral overlap or sidelap). These large overlaps in vertical photos are kept for the following reasons–

(1) To make sure that no part of the ground is left unrecorded.

(2) In order to view the pairs of photographs stereoscopically, only the overlapped portion is useful.

(3) To tie the different prints together accurately, it is desirable that the principal point of each print should appear on the edges of many adjacent strips as possible.

(4) The distortions caused by the lens and by the tilt, and the relief displacements are more pronounced in the outer part of the photograph than near the centre of each photograph. If the overlap is more than 50% these distortions and displacements can be overcome while constructing the maps.

(5) Due to the overlap, each portion of the territory is photographed three to four times. Hence any picture distorted by excessive tilt or by cloud shadows, etc. can be rejected without the necessity of a new photograph.

(6) If the flight lines are not maintained straight and parallel, the gap between adjacent strips will be left. These can be avoided by having side lap.

(7) Objects can be viewed from more than one angle in the stereoscopic examination if sufficient overlap is provided.

Question 3. What is tilt distortion ? What are the advantages and disadvantages of photogrammetric mapping ?

Answer : TILT DISTORTION, ADVANTAGES & DISADVANTAGES OF PHOTOGRAPHIC MAPPING

Tilt Distortion : Tilt distortion or tilt displacement is defined as the difference between the distance of the image of a point on the tilted photograph from the isocentre and the distance of the image of the same point on the photograph from the isocentre if there had been no tilt.

If a terrain is photographed, once with a tilted photograph and then with a vertical photograph both taken at the same flight altitude and the same focal length, the two photographs will match at the axis of the tilt only. The image of any other point, not on the axis of the tilt, will be displaced either outward or inward with respect to its corresponding position on a vertical photograph.

In vertical photographs tilt causes distortion, both in respect of distances and directions. Where the angle of tilt exceeds 4^0, the photographs are unsatisfactory for mapping and required to be satisfied by means of projection printing or by the Canadian High Oblique Rectifier or by the use of other equipment. The Smith three axis automatic control, which is a gyrostatic device for

stabilising the aircraft, reduces tilt and keeps it within 15 minutes angle. In some automatic pilot systems, the course of the aircraft is controlled by a ground radio station so that flight lines are parallel to each other, and take the form of concentric circle of large radii.

The introduction of the Doppler equipment as an air borne navigation aid, giving the ground speed, angle of drift, and distance flown, is a major advance towards ensuring accuracy and controlled photography.

The amount of tilt can be roughly estimated, by a simple device known as Stevensons template or by means of the more complicated Thompson's Comparator. The contour tracer and other instruments can also be used.

Advantages and Disadvantages of Photogrammetric Mapping : *The main advantages of the compilation of topographic maps by using aerial photographs over ground method are as follows–*

(a) The speed of compilation, (b) the reduction in the amount of control surveying required to control the mapping, (c) the high accuracy of the locations of plannimetric features, (d) the faithful reproduction of the configuration of the ground by continuously traced contour lines, and (e) the freedom from interference by adverse weather and inaccessible terrain.

Also by the proper selection of flying heights, focal lengths, and plotting instruments, and by proper placement of ground control, photogrammetric mapping can be designed for any map scale ranging from 1 in 250 down to 1 in 250,000 and smaller, and for contour interval as small as 15 cms. Because of the wealth of details that can be seen in a spatial model, the resultant photogrammetric map will be more complete than will a comparable map produced by ground method.

Disadvantages : Among the *disadvantages of mapping by using aerial photographs* are the following –

(1) The difficulty of plotting in areas containing heavy ground cover such as high grass, timber, etc.

(2) High cost for mapping areas 2 hectares or less in extent.

(3) The difficulties in locating positions of contour lines in flat terrain.

(4) Necessity for field editing and field completing where spot elevation has to be measured in flat terrain.

(5) Names, symbols, height, etc. do not appear on photograph which make identification more difficult.

(6) Most people are familiar with maps and find them easy to read. The air photographs on the other hand show lot of confusing details not normally shown on maps.

Question 4. Write short notes on :

(a) Stereo-photogrammetry ; (b) Mosaic ; (c) Crab and Drift ; (d) Phototriangulation ; (e) Photo interpretation.

Answer : STEREO–PHOTOGRAMMETRY, MOSAIC, CRAB & DRIFT, PHOTOTRIANGULATION AND PHOTO INTERPRETATION

(a) **Stereo-Photogrammetry :** Stereo-photogrammetry or stereo-photographic surveying

which consists in taking stereoscopic views of the surface features in pairs at the ends of a base line. The two exposures must be made with the photographic plates in the same vertical plane. This can be done by taking the two photographs with the vertical planes of collimation of the cameras at right angles to the base line.The length of the base line, the horizontal distance between the principal planes usually lies between 30 and 120 m. It is measured by tape or by the theodolite and stadia method.

(b) **Mosaic :** Vertical photographs look so much like the ground that a set can be fitted together to form a maplike photograph of the ground. Such an assembly or getting of a series of overlapping photographs is called a mosaic. To a varying degree of acuracy, a mosaic is a map substitute.

When the photographs are carefully assembled so that the horizontal control points agree with their previously plotted positions, a controlled mosaic is obtained. A mosaic which is assembled without regard to any plotted controlled is known as uncontrolled mosaic.

Various features appear as realistic photographic images on a mosaic.

(c) **Crab and Drift :** *Crab* is the term used to designate the angle formed between the flight line and the edges of the photograph in the direction of flight. If the focal plane of the camera is not squared with the direction of flight at the instant of exposure, the crab is caused in the photograph. Crabbing should be eliminated since it reduces effective coverage of the photograph.

Drift is caused by the failure of the photograph to stay on the predetermined flight line. If the drifting from the predetermined flight line is excessive, reflights will have to be made to avoid serious gapping between adjacent flight lines.

(d) **Phototriangulation :** In the early period of aerial photogrammetry the usual practice was to determine the co-ordinates of the four points for each aerial photograph by methods of ground surveying. At present there are methods for densification of geodetic points (both of planimetric and vertical control) by phogrammetric techniques at office. This makes it possible to obtain, given two or three points at the ends and at the middle of a strip, a dense net of reference photogrammetric points.

The method of obtaining planimetric co-ordinates relies on the property of vertical aerial photographs to preserve the directions to the points on the ground drawn from the principal point of an aerial photograph without appreciable distortions. This method is known as phototriangulation.

(e) **Photo Interpretation :** In order to adequately convert a photographic plan to a topograpic one in terms of conventional signs it is customary to topographically interpret the aerial photographs i.e. to identify the topgraphic features on the ground from their photo images. It may not be feasible to fully interpret such photographs by office work, so it is necessary to go into the field to identify the object of an aerial survey on the ground. This process is known as field interpretation. Office work interpretation employs : (1) the geometric properties of the objects, viz. their shape and size; (2) their optacal properties in particular, their property to differently reflect light; (3) the mutual position of the light and shadow ; (4) the natural relationship between objects and phenomena ; (5) special reference obtained by detailed interpretation of selected small areas.

To speed up interpreting the air photos magnifying glasses with the magnification up to 5× and stereoscopes that make it possible to obtain a three-dimensional representation of the area from a stereo pair may be used. When examined stereoscopically, the aerial photographs can be interpreted more adequately.

CHAPTER VI

SETTING OUT & MISCELLANEOUS SURVEY

The mine surveyors have often to deal with the laying downn on or belowground of details and directions that have been either drawn on plans or calculated from the result of a survey for excavating and constructional purposes. The survey work may sometimes involve simply for checking purposes. Drivage of tunnels and cross-measure drifts, both inclined and horizontal, setting out underground roads, setting out building foundations, checking verticality of sinking shafts, checking verticality of tall chimneys, ensuring coincident verticality of pillars in contiguous seam, installing headgear and winding engine, locating the site for borehole or shaft, joint surveying etc, are the examples. The surveyor may be called upon to make the necessary preparations and drawings for such works.

ENSURING COINCIDENT VERTICALITY OF PILLARS

To ensure the coincident verticality of pillars and galleries and a minimum thickness of parting in contiguous seams the following procedure should be adopted :

A projected layout as submitted to the D. G. M. S. is prepared and the cross connections between the seams which may be either by cross-cuts or staple pits or properly graded roadways. Theodolite traverse from main haulage road is carried to each of the working sections of the seams and a number of stations are marked permanently in the roof of each of seam workings either by points or by roof plugs. These are then plotted on the same plan showing different seams drivage by different colours. These survey lines form the base for further drivage of galleries. Measurements are made along the existing roads, and the centre line of new roads are marked as long as possible on the roof or roof supports by a dial to start with. A short drivage of 4 to 5 m is made in the approximate direction and then a theodolite is set up and levelled at intersections of the centre lines of roads. The centre line of the new drivage is marked on the roof adjusting the angle between the centre line of the existing road and the new drivage as determined from the development plan. The deep and rise galleries should only be drivien from one side either from dip or rise direction only. When the gallaries have been formed round the pillar to veryfy the accuracy any if any slight discrepancy exists the same shall be rectified in further gallery drivage and plotted on the same plan. It is preferred that bottom section (workings in lower seam) is kept one row of pillars in advance of the upper seam working.

To verify the survey measurements, vertical borehole at every gallery junctions are put.

These boreholes shall indicate the correctness of survey measurements and also thickness of parting betweeen the workings in contiguous seams. Correlation survey between the upper and lower seam workings with a theodolite will indicate the accuracy of the work and spot level at regular intervals along the galleries of each seam. The difference of level between roof of the lower seam and floor of upper seam workings will indicate the thickness of parting between them. Sometimes staple pits are made between the seams which may be used for verifying survey measurements and to facilitiicate ventilation and transport purposes.

JOINT SURVEYING

Let the workings of two adjoining collieries have reached within 30 m of a common boundary necessitating a joint survey to be made. The workings of one colliery are accessible through inclines and those of other through shafts. In such a case the survey work consists of three parts–

1. Surface survey.
2. Correlation through incline and shafts.
3. Underground traverse and location of barrier galleries.

The relative position of shaft, incline, boundary pillars, etc. should be accurately detrmined and for the purpose two points are fixed near the entry of two collieries from a nearby trijunction post and triangulation survey comprising of well-conditioned triangles in the area is carried out. A suitable base line and a check base are carefully measured by a steel tape and necessary corrections for temperature, standardisation, sag, slope, tension, etc. are applied to determine their accurate lengths. Angles of each triangle are very carefully measured with a pre-adjusted theodolite. The true meridian may also be determined by observation of circumpolar star at equal altitudes. The co-ordinates of triangulation stations are then calculated.

Surface traverses are run from triangulation stations to locate boundary pillars, incline, shaft, etc. which may be closed either on triangulation stations or on themselves.

In case of the mine which is accessible through inclines, the correlation of underground workings consist of carrying over a surface traverse to underground workings through inclines directly and closing the same on nearby triangulation stations previously fixed by the shortest available route.

The correlation of underground and surfacae surveys in case of the other mine worked through shafts is a tedious job and consists of suspending two plumb bobs one in each of the shafts and the line joining the wires forms a common base for surface and underground survey. By carrying out surface traverses from previously fixed triangulation stations, co-ordinates of underground survey stations are calculated. Two or more plumb wires in each shaft may be taken to ensure accuracy in work.

Underground traverses in both the mines with a theodolite are then carried out from the points fixed by correlation survey, to the common boundary by the shortest route and as far as practicable close to the galleries near the boundary and closed polygonally. Subsidiary survey

are then accurately carried out from the main traverse to locate the exact ends of drivages.

Co-ordinates of underground traverse and subsidiary survey stations are calculated and the positions of the traverse both surface and underground in the two collieries are plotted on the same mounted paper with the same origin. The galleries, the boundary pillars, etc. are accurately plotted on the plan. In this manner the surface features,boundary pillars and underground workings of both mines towards the common barrier and shafts, inclines, etc. are marked on one plan.

The joint survey should be performed with utmost precision and precautions by the two surveyors of the two mines. Bookings should be made in two note books which should be signed by the two surveyours. All calulations for co-ordinates should be made separately by them and checked jointly.

SURVEY WORKS FOR RECONSTITUTED MINE

Following the nationalisation of Coal Mines small mines worked by different owners having multi-seam and multi-section workings at various stages are to be reconstituted forming a large mine. A surveyor may be asked to carry out necessary work and prepare plans and sections of the reconstituted mine. The factors to be taken into account while preparing such plans and sections and the difficulties likely to be encounterd are as follows :

Factors to be considered : Modern methods of precision surveying combined with correlation with the National Grid may remove many of the dangers in mines, especially those arising from imperfect fixing of boundaries or important correlation between the surface and underground surveys. The following facotrs shold be considered while preparing plans and sections of reconstituted mine–

1. Previously coal has been worked for many years, the workings were first confined to the outcrops of the seams and afterwards at increasing depths and the workings are literally honey-combed with abandoned areas.
2. There are extensive workings of which no plan or records have been preserved.
3. The standard of surveying was extremely low in many cases and no reliance can be placed on plans made during that period even fairly recent days.
4. Whether workings are accessible or not for surveying purposes.
5. Whether plans are plotted by co-ordinate system and the same is corelated with the National Grid.
6. Whether joint survey plans of nearby mines are available.
7. Adequate facilities for accurate instruments and trained personnels.

Difficulties : The following difficulties may be encountred –

1. Inaccuracies of old plans.
2. Non-availability of old plans.
3. Inaccessibility of the old working, due to waterlogging, heavy roof fall, etc.
4. Narrow workings, zig-zag roadways, low heights and workings in most hapazard manners.

5. Missing of permanent survey stations and Bench Marks belowground.
6. Non-availability of modern instruments, equipments and trained personnels.
7. Difficulties in correlating the area with National Grid system.

PLANS AND SECTIONS FOR OPENCAST PROJECT

Opencast surveys have their specific features like mechanisation for stripping and mining operations and involve fixing of control points, surveying of opencast benches which rapidly advance, laying roadways for heavy machineries, measuring stockpiles, maintaining O.B. records and extending survey plans and sections regularly.

For primary control the area should be covered with triangulation. A permanent base line should be 60–100 m in length laid along north-south or east-west direction on a fairly level ground not far off from the quarry and which should remain undisturbed during the quarrying operations. Angular measurements from the fixed base line should be taken and measuring the base accurately, all sides of the triangulation system should be computed. The plan of the area should be prepared from particulars available by triangulation.

Before starting the quarry, large scale plans of the area and sections should be prepared showing the contours at 1.5 m intervals and surface features like building, roads, etc. within and around 30 m on all sides of the quarriable limit.

Plans showing lines drawn through places having equal thickness of seam and overburden have to be prepared for estimating the total quantity of overburden to be removed and to decide the quarriable limit.

Plans showing the surface contours and grid lines should be prepared for periodical measurement of the excavation of overburden and coal.

Necessary plans for drainage, roads for movement of heavy machineries, electrical transmission lines should also be prepared beforehand.

The measurement of valume forms an important part in opencast mines. The conventional method for measurement of valume involves :

1. The study of contour plan, and
2. The movement of bench height.

For both these operations tacheometric survey may be suitable and convenient.

Volume surveys maybe done by –

(i) cross-sectional areas, (ii) contours, (iii) spot heights.

The volume of overburden removed during a particular period is obtained from one of the following –

1. Prismoidal formula
2. End area formula
3. Sympson's rule.

SURVEYING WORKS FOR LOCATING BOREHOLES FOR DEWATERING

The procedure to conduct the survey for dewatering old workings by putting boreholes from surface includes the following–

1. To study all old plans thoroughly.
2. To study the abandoned mine plans of the area to know the extent of old workings.
3. In case of thick seams, horizon of the workings needs to be examined with the help of spot levels and stratum contours.
4. Geological disturbances, like faults and dykes need special attention.
5. The accuracy of old plan must be checked. The extent of working may be different from those indicated on plans.
6. To correlate the surface and underground workings, where accessible, so that the accuracy of the old plans can be checked.
7. To establish the exact extent and geometry of old workings, by putting a number of boreholes from the surface.
8. To study the surface contours and underground spot levels, general direction and dip of the strata, section of the working seam, section of the strata from the surface upto the seam, and to find any substantial variation in the thickness and character of strata.

A theodolite traverse survey should be made to check the position of permanent structures shown on the plan to prove the accuracy of the plan. A suitable point on the dip side of the waterlogged working belowground (not over the standing pillars) should be marked on the plan and the position of the point should be located precisely on the surface by means of a theodolite survey. The point thus established at surface should be used to put a borehole upto the old working at its dip sides for dewatering purpose.

Factors to be considered :

1. A careful assessment of the danger of inundation should be made beforehand.
2. The danger of inundation exists mainly on account of the unreliability of the old plans. The extent of working may not have been shown fully on the old plan.
3. The depth of the working from surface to be ascertained.
4. The old workings could be connected to the waterlogged quarries or fault extending to surface connecting a river bed or pond or lake, etc. in which case dewatering by borehole from surface may not be effective.
5. The point for putting borehole at surface should be located at clear space without disturbing the surface structures and where maximum facilities for pumping are available. The depth from surface should also be minimum.

ERROR IN MARKING POINT FOR BOREHOLE

Let a borehole is put on the rise side of the property for stowing purpose which has gone in virgin area instead of a gallery junction due to inaccurate survey done while giving the mark for borehold. Mine openings are equipped with a shaft and a fan drift.

The survey work to ascertain the error while giving mark for borehole at surface and to connect the borehole by driving a heading from existing rise-most gallery underground consists of –

Surface Survey : A small triangulation survey comprising a series of interlacing triangles forming an accurate framework for the survey is carried out. A base line is accurately measured and the angles of each triangles are measured precisely by a theodolite. The precise position of the borehole marked at surface is ascertained by measuring the distance and bearing from a suitable triangulation station. Co-ordinates of all triangulation stations and also of the borehole point at surface are calculated.

Correlation Survey : The survey is commenced from one end of the line in the surface triangulation in close proximity to the inclined entrance or fan drift. A traverse survey is carried out through the drift and the connecting roads extending from the bottom of that drift to the plumb-line which is suspended in the shaft. The travarse is run through the rise-most area underground including the point at the gallery-junction where the borehole was intended to meet. The survey is also carried out from the base-line to the plumb-line over the surface.

The co-ordinates of all the survey stations underground are calculated.

As the borehole put on surface has gone in virgin area instead of the gallery junction, the co-ordiantes of the two points, the borehole point at surface and the gallery junction point where the borehole was supposed to meet, will not agree. The error can be found out from the difference of co-ordinates between these two points.

Length of line (from the gallery-junction to the borehole point put in virgin area)

= Error in latitude x sec θ Where θ is the bearing of the line joining these two points which can be known from the formula, $\tan^{-1}$ (Diff. in dep ÷ Diff. in lat).

The horizontal direction of the drivage as determined from the co-ordinates between the existing rise-most gallery and the borehole mark projected vertically below is set out with a theodolite. Depending on the prevailing conditions the surveyor secures at least three points, solidly secured in the roof with temporary pegs spaced not more than 2–3 m apart. The centre line of the heading is extended by the assistants as the drivage progresses. Plumb bobs are run from the fixed points and when they have come to rest the men take position behind these so that all three plumb-lines overlap. The obtained direction is projected into the face and marked with chalk. In order to achieve high degree of accuracy the centre line should be checked at regular interval with the help of a theodolite.

As the great importance is attached to the accuracy of the work, grade lines should be given at the sides of the heading in which the required gradient may be instrumentally controlled. Periodical check should be provided by levelling the drivage with a theodolite as the work proceeds.

SURVEY WORK FOR INSTALLING HEADGEAR

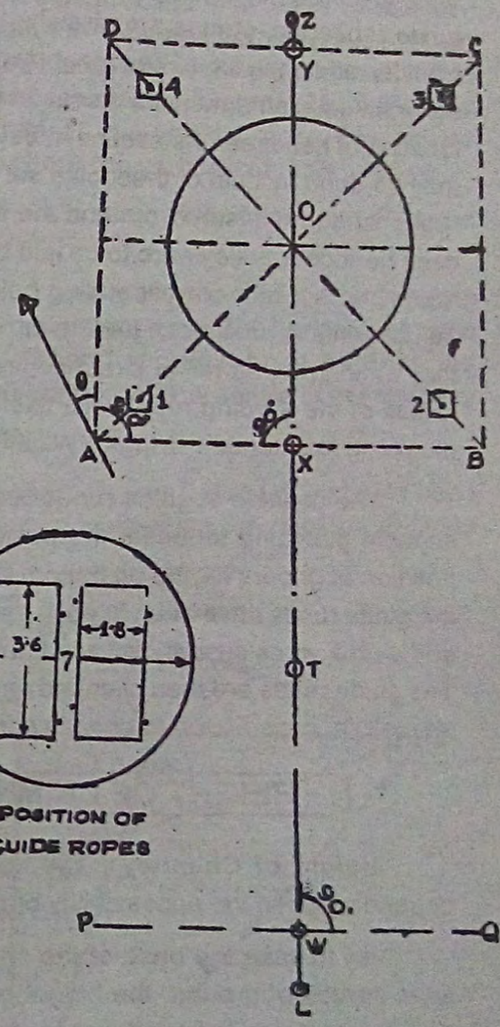
A disused shaft has to be brought into use and a winding engine installed.

In order to install a new headgear, pulleys, guides and the winding engine the following

survey works are involved–

1. To fix the centre of the old shaft and positions of HD. Bolts for the new headgear legs.
2. To fix the centre line of the winder.
3. To fix the positions of winding ropes.

A survey is carried out from some reference point. The positions of the old headgear, details of shaft, etc. are included in the survey. An enlarged plan to a scale of 1 cm = 10 m is drawn showing details of shaft, etc. thus collected. The proposed site of the new winder, positions of foundation bolts, positions of the legs of new headgear and their foundation bolts, position of guide ropes, winding ropes, headgear pulley with plummer block positions, etc are also shown on this plan.



Marking the positions of the HD. Bolts for New Headgear foundation : Let 1, 2, 3, and 4 are positions of the old HD. Bolts. The shaft is covered with girder and strong wooden planks temporarily. A theodolite is set up over station 1 and point 3 is sighted. The distance 1 to 3 is measured carefully and half of this distance is marked which is the centre 'O' of the shaft. This is checked from the plan. Line 1–0–3 is extended bothways. The instrument is set up on stations 2, 3, and 4 and the centre point 'O' is checked in a similar manner. The OA is measured from the plan where A is the centre point of the HD. Bolt for one of the legs of new headgear. After fixing the point A, the instrument is set up at A and the angle SAD is measured where S is the reference station. The point D is marked by taking accurate measurement AD from the plan. The angle DAB = 90° is set out and the point B, the centre point of another leg is marked.

The distance AX is measured and the point X is marked on the centre line of the shaft. The instrument is set up at X and the angle AXO = 90° is measured. The stations Y and Z outside the permeter of the shaft are fixed. The line YXL is set out.

Points M and N are fixed on lines AD and BC respectively by means of the instrument at B and A, and distances AM and BN taken off from the plan so that MON is at right angles YOX.

From the plan, distance OW (the distance from shaft centre to the centre of the winding

drum) is taken off and the point W is fixed accurately. The instrument is set over the point W and angle OWQ = 90° is measured. The line QW is extended to P and thus the centre line of the drum PWQ is obtained.

Fixing up Position of Winding and Guide Ropes : To layout the position of winding rope, guide ropes etc. strings are stretched between YOW and MON and their directions are then transferred on the top of headgear by plumbing from stretched strings and the lines are marked by white paint straight lines. Location of these two lines which forms the basis of all survey for location of headgear installation, must be done with great accuracy and if necessary, checking may be done by help of theodolite set over extended range of lines YW and MN. The centre of ropes is then measured out and the headgear pulley position is correspondingly adjusted. In case headgear pulleys are to be laid out, the position of plummer blocks for headgear pulley is measured out and corresponding holes are drilled and headgear pulleys are installed. Slight adjustments in position of headgear pulleys may be done by adjustable nuts and oval holes so that winding ropes fall in exact centre of plumb line. Due consideration may be given for the radius of the winding rope while fixing position of headgear pulleys. This may be checked by plumbing to extended strings YW.

The position of guide ropes is also fixed by measurement from the above base line and suitable plumbing to locate the position of guide ropes. Plumbing is also done to locate the position of griders for the pit bottom landing and for girder for positioning the guide ropes. After the guide ropes have been fitted in their position they may again be checked by help of base line and plumb wires suspended and final adjustment in position of guide ropes is done if required. The guide ropes are then clamped and tested in slow runs. These ropes may be straightened by a small jim crow on top of cage in case slight curving occured during their installation.

HEIGHT OF CHIMNEY AND CHECKING VERTICALITY

Height of Chimney : The height of a chimney canbe determined in number of ways depending upon the accessibility or otherwise of the base of the chimney.

(a) In case the base of the chimney is accessible and instrument can be set on a fairly level stretch of ground, the height of chimney can be determined by setting a theodolite at a convenient distance from the chimney and the angle of elevation (θ) to the top of the chimney and the instrument height h_1 is measured. The horizontal distance (d) between the theodolite station and base of chimney is also measured. The height of the chimney is obtained by the formula $h = d \tan \theta + h_1$. If the ground is not level, the difference in level between the theodolite station and base of chimney is found out by levelling and height of chimney is obtained by adding or substracting the level difference.

(b) In case the base of chimney is inaccessible , a level base line AB is selected at a convenient distance in line with the chimney and its length is measured. The vertical angles at A and B are measured. Then the height of the chimney above the level line of sight $= h = \frac{AB}{\text{Cot } A - \text{Cot } B}$

The height of the instrument should be adjusted and the difference in level between the instrument

station and chimney base determined to obtain the total height of the chimney from its base.

Checking the verticalityof a Chimney : The verticality of a chimney may be checked in the following manner –

(a) *Approximate Method :* A theodolite is set over a station A and after levelling carefully the mid-point of the chimney at its bottom is sighted. Both the body and vernier clamps of the horizontal circle are tightened and the telescope is rotated slowly in a vertical plane to sight the chimney top. If the chimney is vertical, the vertical hair of the telescope diaphragm will be coincident with the mid-point of the chimeny at its top while it is moved in a vertical plane. The similar procedure should be asdopted from a second point B to check the verticality in this position.

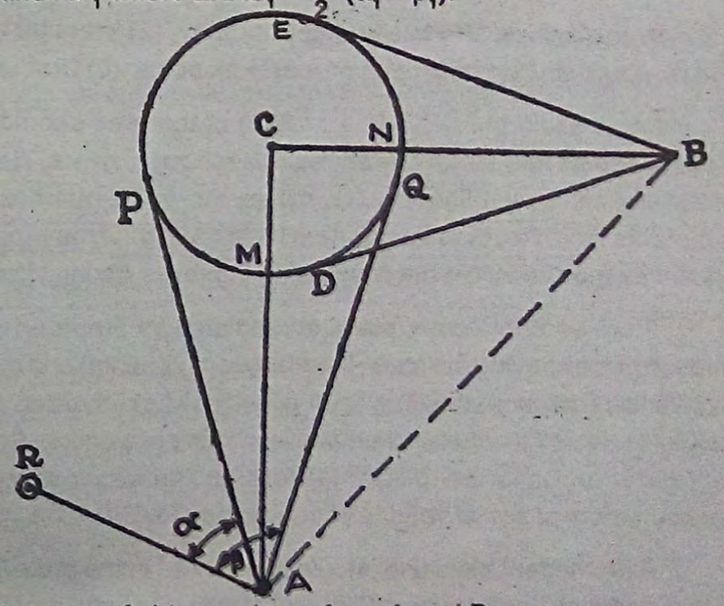
(b) **Accurate Method** : Let C is the centre of the chimney at its base, and R is the reference point.

A theodottite is set up over station A about 15 metres from the chimney base. After levelling the instrument, the reference point R is bisected with the vernier zero-zero. The horizontal angles RAP and RAQ are measured where P and Q are the left hand and right hand edges of the chimney at its base. Let these two angles are α and β respectively. Now centre of the chimney must lie along the line AC, where $\angle RAC = \frac{1}{2}(\alpha + \beta)$.

The angles RAP_1 and RAQ_1 is similarly measured, where P_1 and Q_1 are left and right hand edges of the chimney at its top. Let these are α_1 and β_1 respectively. If the centre of the chimney at its top is C_1, it must lie along the line AC_1 where $\angle RAC_1 = \frac{1}{2}(\alpha_1 + \beta_1)$.

.The angles RAC and RAC_1 should be the same if the chimney is vertical otherwise it is out of vertical.

The same procedure should be adopted from a second point B, where BC is aproximately at right angle with AC. Station A should be taken as the reference point in the case. Angle ABC and ABC_1 should be same if the chimney is vertical in this position. Otherwise it has been deviated.



Let AM and BN = Horizontal distance of chimney base from A and B.

CM = Radius of chimney as obtained from the relation $2\pi r$ and measuring the circumference at its base.

Co-ordinates of point C and C^1 relative to A as origin may be calculated by finding the angles CAB and C_1AB, carefully, and the bearing and length of AB accurately. Co-ordinates of points C and C_1 should be the same if the chimney is vertical. But if not, it has been deviated the amount and direction of which can be calculated.

QUESTIONS AND ANSWERS

Question 1. As a survey of an extensive Opencast mine what surveying , levelling and computation work you will make to generate data for accurate determination of coal : overburden ratio during planning and during execution of the project.

Answer : SURVEYING AND LEVELLING WORKS FOR OPENCAST MINE

In order that proper planning may take place it is necessary to collect as much information as possible on the project. The following maps, sections and data are maintained for planning an extensive opencast mine.

Plans : (1) Topographical plan with surface contour at close intervals. (2) Borehole plans, showing altitudes, mineral thickness and values. (3) Isopachytes, deposit thickness, ovrburden ratio, grade. (4) Hydrological plans showing the free ground water levels and confined ground water. (5) Reserve plan. (6) Mining plans showing blocks to be worked.

Sections : (1) Geological sections (2) Bench levels. (3) Exposed reserves. (4) Ground water tables.

Data : (1) Irregular topography. (2) Irregular Physical Characteristics of mineral and overburden. (3) Limitations on waste disposal. (4) Drainage.

Irregularities : (1) Irregular topography. (2) Irregular Physical Characteristics of mineral and overburden. (3) Limitations on waste disposal. (4) Drainage.

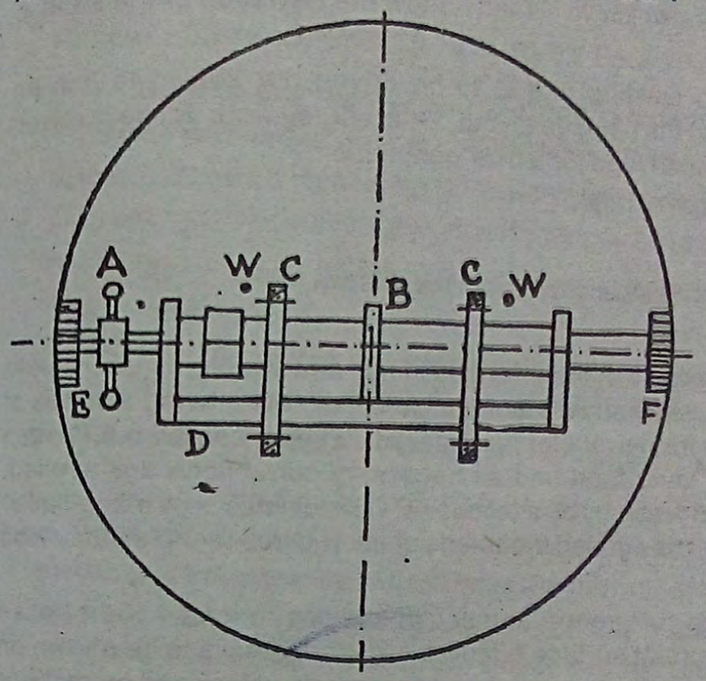
Large scale plans (R. F. 1 : 1200) of the area should be prepared showing contours at close intervals and surface features like railways, rivers, roads, buildings, boreholes, inclines, telephone and power lines, faults, dykes, etc. in the quarriable limit and 30 m on all sides of the quarriable limit. Residential quarters within 500 m of the proposed quarry boundaries should be shown as they lie within the range of flying rocks during blasting.

From borehole data, plans should also be prepared showing seam/deposit (Line drawn through places having the same true thickness of seam) and overburden isopaches. These assume significance when estimating total quantity of overburden that is to be removed and are also helpful to decide the quarry limit at evey phase of working. Presence of old workings showing the dimension of galleries, pillars left etc. in the seam should also be shown on the plan for measurement of actual volume of coal removed.

A permanent base line should be marked in the area. The base line should be nearly NS or EW in direction and between 60 m and 100 m in length, preferably on level ground not far from the quarry and located in area, which will be undisturbed by quarrying operation or movement of machinery. On the contour plan grid lines, 30 m apart, parallel and at right angles to the base line should be marked and extended 100 m on all sides of the quarriable area. On the ground, the junction of the grid lines should be shown by masonary pillars. The base line should be similarly marked on the ground by masonary pillars. Such a plan showing surface contours and grid lines is necessary for periodical measurement of the excavation of the overburden and coal.

Question 2. A doubt has arisen about the verticality of a 4.8 m dia. 210 m deep working shaft. State briefly how you would carry out a survey of the shaft. What instrument would you require, how you would take the measurements and interprete the results ?

Answer : SURVEYING FOR DEVIATION OF SHAFT



A : Screw jack adjustment

B : Sight vane

C : Thumb Screw

D : Graduated rod

E : Capping (Swivel end)

F : Capping (fast end)

W – W = Wires

To ascertain the amount and position of any deviation from vertical of any shaft the following methods may be adopted –

(1) By suspending a single wire at the centre of the shaft and taking offsets at regular interval from the shaft sides.

(2) By suspending two or more wires and taking offsets to points in the shaft.

(3) By Dyson's device and taking offsets from its zero graduation fixed at the shaft centre.

Dyson's device :

For accurate results this device is used for shaft detail surveying.

It consists of a long length of pipe, strong enough to carry its own weight without bending. The pipe can be jacked between the shaft walls by screwing an inner length of pipe fitted into the long pipe. The upper frame carries a graduated wooden rod which can be adjusted for position by means of thumb screws. The rod is fitted with the scale at each end, and a sighting vane of its centre.

Precedure :

The centre of the shaft is fixed at the surface and two plumb wires are suspended in the shaft. The perpendicular distance from the centre of the shaft to the plumb plane and from the centre of the plumb plane to one of the wires are measured. The plumb wires serve to set the device with its zero graduation at the correct position of the shaft centre.

The apparatus is now jacked against the shaft walls in the required position where the first set of offsets is required. The rod is adjusted by sighting the wires against the scale for maintaining the correct offset distance and also for making it parallel to the plumb plane. Its central zero

graduation should also be in correct position relative to the wires. The zero point is then made to occupy the shaft centre from which offsets are taken to the shaft sides at least for two per-fixed positions at right angles to each other. The process is repeated at regular intervals of 5 metres.

All the informations in these offset surveys, i.e. vertical positions of shaft centre and offsets taken therefrom are carefully recorded. Section of the shaft at different horizons is then plotted on a suitable scale from the collected data which will clearly show the deviation of the shaft in any particular direction.

Question 3. A new incline adjacent to existing one is to be driven out 200 m to join an underground dip roadway for straightening the haulage layout. What survey work will be involved to ensure the direction and gradient of the drivage for a true connection ?

Answer : DIRECTION AND GRADIENT OF DRIVAGE

1. A surface traverse.
2. Correct survey through the existing incline by direct traversing.
3. Underground traverse.

The relative position of the existing incline, important buildings and structures etc. should be determined accurately. For this purpose, a small triangulation survey comprising a series of interlacing triangles forming an accurate framework for the detailed survey is carried out. A base line and check base are measured by invar tape and all necessary corrections are applied. Angles of each triangle are then measured precisely by a theodolite. Co-ordinates of all triangulation stations are calculated and plotted on suitably grided mounted paper. The true meridian may also be determined for the purpose.

A closed surface traverse is then run from triangulation stations to locate the precise positions of the existing incline and important surface features and other details to be shown on the plan in relation to underground workings. The survey after necessary checkings and corrections are plotted on the triangulation framework.

The correlation is carried out by making a precise theodolite traverse direct into the underground workings, starting from some one line of known bearing in the surface triangulation and making a return traverse to the initial reference point so as to close the survey ploygonally. Subsidiary survey are then accurately carried out from the main theodolite underground traverse to locate the exact point at dip roadway, the end point of the incline to be driven from surface. Co-ordinates of underground traverse and subsidiary survey stations are then calculated and detailed plotted on the same mounted paper.

Studying the surface contour and spot levels at dip roadway from the existing mine plans the approximate difference of levels between the surface and underground dip roadway is found out and the horizontal length of the new incline 200 m long calculated. Considering the availability of space, purpose of the incline, etc. the most suitable position of the new incline at surface at a horizontal distance from the dip roadway thus calculated is marked on the new plan. The point is then precisely located at surface connecting the same with the main triangulation stations by a theodolite survey. The co-ordinates of the point from which the centre line of the new incline is to be set are calculated and the length and direction of the incline are determined, Exact difference of levels between the two ends of the incline is found out by carrying out levels from the point at the surface to the dip roadway below ground. The exact length of the incline may then be found out.

If desired the surface point may be shifted for convenience but in such case a re-survey by a theodolite and by taking levels, the co-ordinates of the point, and direction and length of the incline should be determined.

During drivage of the incline both the direction and gradient must be instrumentally controlled.

Question 4. Some of the developments in conventional and new techniques are given below. Describe them briefly.

(a) EDM and Gyroscopic method (b) Electronic system of co-ordination determination (c) Motorised survey instrument (d) Computers in Mine Surveying.

Answer : DEVELOPMENTS IN SURVEY TECHNIQUES

(a) **EDM and Gyroscopic Method :** Modern conditions in mine surveying require instruments for the limits of error stipulated for each time of survey. Because of introduction of electromagnetic distance measuring (EDM) instruments (such as Geodimeter, Tellurometer, Mekometer, etc.) and gyrotheodolites, significant changes have become possible in the standards attainable.

With EDM instruments, primary traverse nets can be established in modern mines with sides of a 500m or more through long, straight main transportation and ventilation drifts. The time taken for underground precise traverse will be reduced by the application of these new instruments. There is now a wide choice of instruments capable of measuring distances up to 3 km with a mean square error better than 10 mm.

Correlation surveys no longer need the time consuming operation of plumb–wires hanging in the shafts, but can quickly and accurately be carried out by the use of optical plumbing and gyroscopic transference of base lines. Several models of gyrotheodolites are now available, and most are able to provide an average accuracy better than 30" in azimuth. An optical plummet utilising a laser beam has been developed by which co-ordinates can be transferred underground from the surface.

The Wild GAK I gyroattachment and the Wild D1.10 Distomat are the instruments which have been modified for underground safety requirements and the accuracy attainable with the latter having a mean square error of 10 mm, falls well within the prescribed limits with traverse legs longer than 50m. Several instruments also measure angles to a few seconds and both angular and linear measurements can be obtained with same instrument from one setting. Accurate distance and circle measurements are displayed in digital read-out form and/or on punched tape directly in the field in some instruments.

(b) **Electronic System of Co-ordination Determination :**

Computation in most surveys may either be carried out on the surface of a reference spheroid using the mid-latitude or other formulae of plane trigonometry. The latter method is preferred by all but those having access to an electronic digited computer.

In precise work, the observed data–angles and sides–may be adjusted for consisteney prior to the computation of final co-ordinate values. Semi–rigorous or least squares adjustment may take place according to circumstances. Also, either the method of condition equations or of observation equations may be used. Generally, the method of observation equations and variation of co-ordinates is preferable because of its simplicity and versatility. The semi-graphic method of combining the adjustment with the computation of co-ordinates has much to command it, whilst an analogue computer may be used to handle the semigraphic method.

In spheroidal computation the spheroidal lengths are used in the computation directly, whereas in projection work the necessary scale factors have to be applied.

(c) **Motorised Survey Instruments :** The Fennel KT–1 gyrotheodolite, wild GAK instrument and some instruments having the gyro are motorised. Power for the gyromotor is obtained from a 12–Volts lead plate battery and converted to three phase alternating current with a frequency of 400 Hz at 110V in a specially designed transistorised generator. The gyromotor is built as a 3–phase induction motor enclosed in a cylindrical case. The gyro rotor spins at about 22000 r.p.m. which ultimately takes the position in a meridian plane.

(d) **USE OF COMPUTERS IN SURVEYING :** The development of mine surveying in recent time, and especially in the last two or three decades of the total scientific and engineering progress, has been associated with the improvement of existing and design of principally novel instruments, systems and techniques of field and office work. The scientific and applied aspects of mine surveying are being developed intensively. Mine surveying problems are now solved with wide use of electronic computers and automatic devices. It is worth to mention the coded theodolites, with direct input of measured results into electronic computers, special-purpose electronic computers for mine surveying, desk calculators, etc. among many achievements in this field.

Electronic computers, large and small, may rely for power both on DC and AC sources in the range from 110 to 220V and on storage batteries. Pocket calculators are particularly convenient in the practical work of a surveyor. They are small in size and use primary or secondary cells. The range of operations of this device is great : they can perform both simple arithmetic operations and complex computations. These calculators make it possible to make calculations without the use of any tables. A pocket calculator can calculate natural (Naperian) and decimal logarithms, trigonometric functions and find reciprocal numbers. They are reliable, simple and easy to use. In addition, they are extremely accurate and consume less labour and time.

The use of computers permits tacheometric surveys to be made by doing away with running theodolite or tachometric traverses that are needed for conventional tacheometric surveys.

Question 5. Two vertical shafts which are about 1000 metres apart are to be connected at a predetermined datum by a straight drive of 3.0m x 2.5m section driven from both sides. Discuss the survey controls to be exercised for the project and methods for executing the work.

Answer : SURVEY CONTROLS FOR STRAIGHT DRIVAGE

The work may be divided under the following heads :

1. A surface traverse including the two shafts to locate the correct position of shaft centres.
2. To locate the shaft centres at the bottom of the shaft.
3. To compute the co-ordinates of the shaft centres and to find out the bearing of the line joining two shaft centres.
4. An underground traverse joining shaft centres through a suitable route, if available.
5. Drivage of the roadway in the horizontal direction controlled by a theodolite.
6. To maintain the gradient of the drivage.

The surveyor should locate the exact point of shaft centres at the surface and at the bottom of the shaft by suspending plumb–wires. A surface traverse and an underground traverse through an available route should be made connecting the plumb-plane at the surface should be found out.

The horizontal direction of the drivage at the bottom of the shaft is set out with theodolite at the predetermined direction with the help of a theodolite and fixed with at least three points, solidly secured in the roof with temporary pegs spaced not more than 2–3m apart. For setting out the direction at the shaft bottom two plumb-lines suspended from the surface at one shaft will be helpful to treat as a reference line.

Depending on the prevailing conditions the surveyor secures plumb–bobs along the centre line of the drivage. As the drivage progresses, the driving crew checks the working for direction. Plumb-bobs are run from the fixed points and when they have come to rest the men take position behind these so that all three plumb–lines overlap. The obtained direction is projected on to the face and marked with chalk. The check should be carried out at all places of drivage :

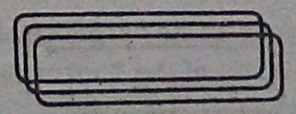
1. When pointing shortholes over the face.
2. While checking the cross–section of the working following shot–firing and loading out.
3. While setting supports.

In order to achieve high degree of accuracy the centre line of the drivage should be checked with the theodolite at an interval of progress of about 15m.

To maintain the gradient of the drivage

A theodolite may be used. A grade line should be established on the sides of the drivage in white wash at a fixed distance usually 1 metre above the floor. Two or three pegs are inserted on the side wall at suitable intervals along the grade line. The theodolite is used to fix the position of pegs in correct gradient. A cotton string is stretched between these pegs and the line is extended and marked by chalk.

As the great importance is attached to the accuracy of the work, periodical check should be provided by levelling the drivage with a theodolite as the work proceeds.



CHAPTER VII

OBJECTIVE TYPE QUESTIONS

THEODOLITE SURVEYING

Fill in the blanks correctly :

1. A theodolite in which its telescope can be revolved through complete revolution about its horizontal axis in the vertical plane is known as ..... theodolite.
2. A standard transit theodolite (Vernier type) consists of the telescope mounted on a ..... axis so that the telescope can be rotated in a vertical plane.
3. The top clamp of a vernier type theodolite is called the ..... clamp and it holds the ..... of the graduated circle.
4. Telescope on modern theodolites are usually the ..... focussing type.
5. The process of setting the theodolite over or under a station mark is known as .....
6. The process of turning the telescope about the vertical axis of the instrument in the horizontal plane is called .....
7. Aplanatism means the absence of ..... .
8. The ratio of the focal length of the objective to that of the eye-piece is known as .....
9. The theodolite should be set on a hill side with ..... down hill and ..... up hill to ensure stability.
10. Parallax is removed by turning ..... .
11. For a check on angular measurements in a closed traverse the sum of interior angles should be equal to ..... .
12. For a check on linear measurements in a closed traverse the sum of ..... should be equal to sum of ....., and sum of ..... should be equal to sum of ..... .
13. The latitude and departure of any point with reference to the preceding point are called ..... of that point.
14. Errors due to unequal graduations on the horizontal plate can be minimised by repeating the measurement on ..... of the horizontal circle and ..... .
15. The upper plate of a theodolite is fixed to ..... .

16. ..... is used for centring a theodolite in mine roadways where velocity of air is high.
17. Modern theodolites are fitted with a ..... which is screwed to one of the standards.
18. The reading of the verniers on the vertical circle of a theodolite must be ....., when the line of sight is horizontal.
19. ..... attachment mounted on a theodolite determines true meridian at any place during day or night with high degree of accuracy in less than 20 minutes.
20. The long sensitive spirit bubble tube attached to the top of one of the standards is known as ..... and it can be centred by means of the ..... .
21. The telescope of the theodolite is supported in the standards by means of an axis called ..... .
22. The ..... stage is a flat base of the theodolite which is threaded to screw the instrument on the tripod.
23. The tribrach and trivet stages are connected through the ..... .
24. The adjustment of theodolite is being very delicate, perfect adjustment is not possible and consequently, very small errors remain unadjusted which are called ..... .
25. The error due to defective centring varies inversely as the .....

Answers : (1) transit (2) horizontal (3) vernier, vernier (4) internal (5) centring (6) swinging (7) spherical aberration (8) magnification (9) two legs, one leg (10) focussing (11) 2n–4 right angles (12) Northings, Southings, Eastings, Westings (13) consecutive co-ordinates (14) various parts, averaging (15) inner spindle (16) optical plummet (17) tubular compass (18) zero (19) zero (20) altitude spirit bubble, clip screws (21) trunnion axis (22) trivet (23) levelling screws (24) Residual errors (25) lengths of sights.

Choose the correct answer from the given alternatives in each of the following :

1. The most commonly method for determining correctly the relative positions of a number of survey points is–
   (a) Chain survey (b) Dial traverse
   (c) Plane table survey (d) Theodolite traverse.
2. Turning the telescope in the vertical plane about the horizontal axis is called–
   (a) Transiting (b) Plunging
   (c) Swinging (d) Both (a) and (b)
3. For measuring a horizontal angle between two lines with better accuracy, the method commonly used is–
   (a) Reiteration (b) Repetition
   (c) Double-sighting (d) Double-centring.

4. A deflection angle is–
   (a) always less than 90° (b) always between 90° and 180°
   (c) equal to the difference between the included angle and 180°,
   (d) always more than 180°.
5. If the graduated and vernier–plates of a theodolite are not concentric the instrumental imperfection is known as –
   (a) Eccentricity of verniers (b) eccentricity of centres (c) graduation error.
6. The horizontal angle between two lines is generally measured–
   (a) Clockwise from forward station (b) Clockwise from the back station
   (c) Counter clockwise from forward station (d) Counter clockwise from backward station.
7. In a theodolite traverse for surveying, the lengths of the traverse lines are generally measured with a –
   (a) Chain (b) Metallic tape
   (c) Steel tape (or band) (d) Invar tape.
8. "Spire Test" is done for adjustment of –
   (a) Plate level (b) Collimation
   (c) Horizontal axis (d) Telescope level.
9. The latitude of a line is obtained by multiplying its length by–
   (a) Sin θ (b) cos θ
   (c) tan θ (d) cot θ
   where θ is the reducing bearing.
10. The most accurate method of plotting a theodolite traverse is by–
    (a) Included angles (b) Tangent method
    (c) Consecutive co-ordinates (d) Independent co-ordinates.
11. The omitted measurements can be computed if they are not more than–
    (a) 1 (b) 2 (c) 3 (d) 4
12. Reiteration method is used for measuring–
    (a) Vertical angles (b) horizontal angles
    (c) distance.
13. Where angular measurements are more precise than linear measurements, the method to be applied for balancing the traverse is –
    (a) Bowditch's method (b) Transit method
    (c) Axis method (d) Graphical method
14. Inaccurate centring of a theodolite is–
    (a) Instrumental error (b) Personal error
    (c) Natural error.

15. If the reading of the verniers on the vertical circle is not zero when the line of sight is horizontal the error known as–
    (a) Index error (b) Residual error (c) Graduation error.
16. Achromatism means the absence of–
    (a) Spherical aberration (b) Chromatic aberration
    (c) Coma (d) Distortion.
17. The most common method of traversing with a theodolite is–
    (a) Continuous azimuth method (b) Double Foresight method
    (c) neither (a) nor (b)

Answer : 1 (d) ; 2 (d) ; 3 (b) ; 4 (c) ; 5 (b) ; 6 (b) ; 7 (c) ; 8 (c) ; 9 (b) ; 10 ; (d) 11 (b) ; 12 (b) ; 13 (b); 14 (b) ; 15 (a) ; 16 (b) ; 17 (b).

SETTING OUT CURVES

Fill in the blanks with suitable words :

1. Gradual and smooth change of direction from one straight to another is known as .....
2. A ..... curve consists of a single arc connecting two points.
3. When a curve of one radius runs into another of different radius bending at the same direction with a common tangent and their centres on the same side is called ..... .
4. Surpentine curve is also known as ..... .
5. ..... is the distance from the tangent point to the point of intersection.
6. If R be the radius of the curve and θ the angle subtended by the chord at the centre, the tangent distance ..... .
7. The sharpness of the curve is designated either by its ..... or by its .....
8. When a curve deflects to the left side of the progress of survey it is called .....
9. When the vertical curve has a convexity upwards, the curve is known as ..... and when such curve having concavity upwards the curve is ..... .
10. The length of vertical curve is obtained by dividing the algebraic difference of the two grades by the ..... .

11. ..... and ..... method is more accurate and for laying out more important curves underground.
12. The super elevation is known as ..... .
13. Supper elevation in cms = ...
14. A circular curve is known as a one, two or five degree curve as its ..... chord subtends at the centre an angle of 1^0, 2^0, or 5^0.
15. The ..... method is more accurate and for laying out more important curves underground this method is adopted.

Answer : (1) Curve (2) simple (3) compound curve (4) Reverse curve or Ogee (5) Tangential distance (6) R x(tan θ/2) (7) radius, degree of curvature (8) left hand curve (9) Summit curve, Valley curve (10) rate of change of grade (11) chords, angle (12) equilibrium cant (13) Track gauge x V^2/gR (14) 20 m (15) chord and angle.

Choose the correct answer from the given alternatives in each of the following :

1. A simple curve is designated by–
   (a) degree of curve. (b) radius of curve.
   (c) both.
2. The necessity of providing a simple curve on a road is to change–
   (a) direction (b) grade
   (c) both (a) and (b).
3. The magnitude of super elevation depends upon–
   (a) radius of curve (b) speed of vehicle
   (c) both
4. The distance from the point of intersection to the mid-point of the curve is–
   (a) rise (b) apex distance
   (c) straight (d) versed sine of curve.
5. Vertical curves are introduced at–
   (a) Change of direction (b) Change of gradient
   (c) None (d) both (a) and (b)
6. The length of long chord and the tangent of a circular curve of radius R will be equal if the angle of deflection is–
   (a) 30^0 (b) 60^0
   (c) 90^0 (d) 120^0
7. If the degree of curve (specified length 30m) is 3^0, the radius of curve is approximately–

   (a) 573 m (b) 1910 m
   (c) 380 m (d) none of the above.
8. The shape of the vertical curve is generally–
   (a) circular (b) parabola
   (c) spiral (d) hyperbola
9. For a design speed of 100 km per hour, the sight distance on the summit of a vertical curve is approximately–
   (a) 150 m (b) 250 m
   (c) 500 m (d) 1000 m.
10. The vertical curve having convexity upwards is called–
    (a) Valley curve (b) Summit curve
    (c) Sag curve.
11. The vertical curve having concavity upwards is known as–
    (a) valley curve (b) summit curve
    (c) crest curve.
12. Transition curve may be a–
    (a) cubic parabola (b) cubic spiral
    (c) both (a) and (b).

Answer : 1 (c) ; 2 (c) ; 3 (c) ; 4 (b) ; 5 (b) ; 6 (d) ; 7 (a) ; 8 (b) ; 9 (a) ; 10 (b) ; 11 (a) ; 12 (c) .

TRIANGULATION, TRILATERATION AND TRAVERSING

Fill in the blanks correctly :

1. The method of triangulation requires a considerable amount of precise ..... measurement with a minimum amount of ..... measurement.
2. Trilateration is based on the ..... that if the three sides (laterals) of a triangle are known, the three angles can be computed.
3. Very large triangle on the earth's surface are spherical and the sum of the spherical triangle is 180^0 plus an amount known as ..... .
4. The ..... is the most accurate and would, generally, form the basis for the surveying and mapping of a whole country.
5. Length of base line in Primary triangulation is ..... .
6. The ..... triangulation consists of a number of points fixed within the framework of primary triangulation.

7. Degree of accuracy in Secondary triangulation is ..... .
8. In ..... triangulation the sizes of the triangles are small and instrument with moderate precision may be used.
9. The length of base line in Tertiary triangulation is ..... .
10. Complex figures in triangulation should not involve more than about ..... conditions.
11. The best shape of triangle in triangulation is ..... with base angles .....
12. The measurement of ..... forms the most important part of the triangulation operations.
13. Measurement of base line by tape in ..... is specially advantageous in rough and hilly country.
14. Correction for sag is always ..... .
15. After measuring the horizontal angles in a triangulation survey ..... adjustment and ..... adjustment are made in order to determine their most probable values.
16. In colliery triangution an accuracy of ..... for calculated distance between two stations should be attained.
17. Sometimes a short base line is measured very accurately and extend it by employing ..... method or by ..... .
18. In triangulation figure, the angle of every triangle should be not less than ..... and not more than ..... .
19. The most common method of determining true meridian in day time is by ..... of the sun.
20. For determining the latitude a ..... is fitted to an ordinary transit theodolite which enables to solve astronomical triangle mechanically.
21. A Geodimeter is based on the propagation of ..... .
22. Generally, the strongest and most economical figure is a .....
23. A subsidiary station is selected near the main station of triangulation as instrument station is known as ..... .
24. ..... stations should be avoided as for as possible in primary triangulation.
25. The group of triangles meant for extending the base line is known as ..... .

Answer : (1) angle, distance (2) trigonometric proposition (3) spherical excess (4) Primary triangulation (5) 5 to 15 Km. (6) Secondary (7) 1 in 50,000 (8) Tertiary (9) 0.3 to 3 Km (10) twelve (11) isosceles, 56°14' (12) base line (13) Catenary (14) negative (15) station figure (16) 1 in 5000 (17) Snellius Rhombus, Double Extension (18) 30°, 120° (19) equal altitude (20) Solar attachment (21) modulated light waves (22) braced quadrilateral (23) satellite station (24) satellite (25) base net.

Choose the correct answer :

1. The figure most commonly used in triangulation is–
   (a) braced quadrilateral (b) quadrilateral with a centre point
   (c) quadrilateral with one diagonal.
2. A satellite station is required when a main station–
   (a) cannot be sighted (b) can not be occupied (c) both (a) and (b).
3. The most accurate method of the measurement of the base line is–
   (a) invar tape method (b) tacheometric method (c) E. D. M. instrument method.
4. The elevated towers are used for–
   (a) instruments (b) signals (c) both (a) and (b)
5. Sometimes a subsidiary station is selected near the main station of triangulation as instrument station which is known as–
   (a) Satellite station (b) False Station
   (c) Eccentric station (d) All above.
6. The correction for standard or absolute length of a base line is–
   (a) Negative (b) Positive (c) Either negative of positive.
7. Correction for horizontal alignment is–
   (a) Negative (b) Positive
   (c) Either negative or positive.
8. The apical angles of the triangles of a Snellius Rhombus should not be less than–
   (a) 20^0 (b) 30^0
   (c) 60^0 (d) 15^0
9. The observation of a circumpolar star at equal altitudes is preferred in colliery triangulation for finding–
   (a) magnetic meridian (b) longitude (c) true meridian.
10. For finding true north the instrument used is–
    (a) Geodimeter (b) Gyro-theodolite (c) Radar apparatus.

Answer : 1 (a) ; 2 (b) ; 3 (c) ; 4 (c) ; 5 (d) ; 6 (c) ; 7 (c) ; 8 (a) ; 9 (c) ; 10 (b).

CORRELATION

Fill in the blanks correctly :

1. Direct transference of azimuth down the shaft by ..... methods are not suitable for shafts exceeding 500 metres.
2. In the majority of cases, correlation has to be carried out down one or more vertical shafts which entails the ..... in the shaft.
3. The most suitable type of wire used in Plumbing the shaft is ..... .
4. In a deep shaft, it is possible that the wire will not come to ..... and it is then necessary to determine its ..... .
5. In..... method the theodolite on the surface must be exactly aligned with the plumb plane.
6. The ..... is formed by the two wires and two instrument stations.
7. The permissible angular error for correlation is ..... of arc.
8. In shaft plumbing methods, the plumb bobs are immersed in buckets of water to cut down ..... of the wire.
9. If the weisbach angle is less than ..... , there is no necessity of measuring sides precisely but the angle should be measured very accurately.
10. The weisbach angles must not exceed ..... to avoid great precision in the measurement of sides.
11. In deep shafts far apart, ... of the verticals make an appreciable difference to the wire interval below ground as compared with their positions at surface.
12. The plumb bobs for shaft surveying should be symmetrically shaped and fitted with vanes to dampen the ..... when immersed in water.

Answers : (1) Optical ; (2) suspension of plumblines ; (3) hand drawn piano wires ; (4) rest, correct position ; (5) coplaning ; (6) Weiss quadrilateral ; (7) ± 2 minutes ; (8) Oscillation (9) 25 minutes ; (10) 20 minutes ; (11) convergence ; (12) twisting motion.

Choose the correct answer :

1. Where entry to a mine is gained by means of a level or drift, correlation may be carried out by–
   (a) weisbach triangle method (b) Coplaning method
   (c) direct traversing method.
2. Permissible limit of error for direct traverse in correlation is–
   (a) ±1 minute of arc (b) ± 2 minutes of arc
   (c) ± 3 minutes of arc.
3. Plumb bobs are normally made of lead which is unaffected by–
   (a) simple pendulum motion (b) air current
   (c) magnetic attraction.
4. Correlation by wires in a single vertical shaft may be done by coplaning method which is also known as–
   (a) weisbach triangle method (b) approximate alignment method
   (c) exact alignment method.
5. Permissible errors of correlation by wires is–
   (a) ± 1 min. of arc (b) ± 2 min. of arc
   (c) ± 5 min. of arc.

6. The plumb bobs must hang freely under water, clear of sides of–
   (a) wire (b) bucket
   (c) instrument
7. With Gyrotheodolites direction of an underground line situated anywhere in a mine can be determined with an accuracy of about–
   (a) 30 seconds of arc (b) 25 seconds of arc
   (c) 12 seconds of arc.
8. To mark the river, lake, tank, pond, etc. on mine plane so as to take precautions against inundation the survey to be done by–
   (a) traversing (b) correlation (c) levelling
9. In Weisbach's method the theodolite is set up slightly out of line forming a small triangle with the two wires and the triangle is known as–
   (a) triangle of error (b) weisbach's triangle (c) isosceles triangle.
10. When the plumb wires have reached the bottom of a deep shaft, the wires should hang free from obstruction or kink and these are checked by–
    (a) moving the wires from the surface (b) passing light from a lamp round the wire from the shaft bottom (c) passing light washers through the wire from surface.
11. The surface survey is connected to the underground survey through–
    (a) a shaft (b) an adit
    (c) both (a) and (b) (d) neither (a) and (b)
12. The survey station belowground is–
    (a) always in the roof (b) always on the floor
    (c) may be in the roof or on the floor.

Answers : 1 (c) ; 2 (a) ; 3 (c) ; 4 (c) ; 5 (b) ; 6 (b) ; 7 (c) ; 8 (b) ; 9 (b) ; 10 (c) ; 11 (c) ; 12 (c).

PHOTOGRAMMETRY

Fill in the blank with suitable words :

1. Photogrammetry may be divided into ..... classes.
2. The system of surveying in which plans or maps are prepared from photographs that are taken from a ground station is known as ..... .
3. ..... is particularly suitable for inaccessible regions, hidden areas, unhealthy regions or wooded country.
4. ..... photogrammetry is not suitable for flat and wooded country.
5. The principle of terrestrial pghotogrammetry is exactly similar to that of ..... surveying.
6. The terrestrial photogrammetry can be divided into (a) .....and (b) .....
7. The photo-theodolite is a combination of 1-second theodolite and a .....
8. A photo-theodolite essentially consists of a ..... of fixed focus type.
9. A circular ..... is mounted on the base of the frame of photo-theodolite.
10. Vertical photograph is useful for..... mapping of great precision.

11. When the axis of the camera at the time of exposure is inclined, the photographs taker. are called .....
12. When vertical and oblique photographs are taken simultaneously by two cameras mounted in the air craft photographs taken are called ..... photographs.
13. The ground control in Aerial photogrammetry is done by ..... method and ..... method.
14. The single lens camera is very simple and robust in construction and free from .....
15. ..... film is invariably used in multi-lens camera.

Answer : (1) two ; (2) Terrestrial photogrammetry ; (3) Aerial photogrametry ; (4) Terrestrial ; (5) plane table ; (6) (a) Plane table photogrammetry (b) Terrestrial steriophotogrammetry (7) terrestrial camera ; (8) fixed focus ; (9) magnetic compass (10) large scale ; (11) oblique ; (12) composite ; (13) pre-pointing, post-pointing ; (14) optical defects ; (15) Panchromatic.

Choose the correct answer from given alternatives :

1. Plotting of maps from photographs taken from air can be done by–
   (a) Radial line method (b) Pre-pointing method
   (c) Post-pointing method (d) both (b) and (c).
2. In case of low oblique photographs, the lilt of the camera ranges from–
   (a) 0^0 to 10^0 (b) 10^0 to 20^0 (c) 20^0 to 30^0
3. The angle formed between the flight line and the edges of the photograph in the direction of flight is known as–
   (a) tilt (b) crab (c) mosaic
4. The failure of the photograph to stay on the predetermined flight line causes–
   (a) drift (b) crab (c) overlap
5. The amount of tilt can be estimated by –
   (a) Stevenson's template (b) Thompson's Comparator
   (c) both (a) and (b) (d) neither (a) nor (b).
6. The terrestrial photogrammetry can be divided into–
   (a) two branches (b) three branches (c) four branches.
7. Terrestrial stereophotogrammetry is a branch of–
   (a) ground photogrammetry (b) aerial photogrammetry (c) aerial surveying.
8. Terrestrial photogrammetry is not suitable for–
   (a) flat country (b) wooded country (c) both (a) and (b).
9. To a varying degree of accuracy, a mosaic is a substitute of a –
   (a) crab (b) map (c) drift
10. The field work of terrestrial photogrammetry consists of–
    (a) Reconnaissance (b) triangulation
    (c) camera work (d) all the above.

Answer : 1 (a); 2(c); 3(b); 4(a); 5(c); 6(a); 7(a); 8(c); 9(b); 10(d)

THEODOLITE SURVEYING

SHORT QUESTIONS

Question 1. What is a transit theodolite ?

Answer : A transit theodolite is one in which its telescope can be revolved or transited through complete revolution about the horizontal axis in the vertical plane .

Question 2. What is Mining theodolite ?

Answer: Mining theodolites are specially used on underground traverses.Such theodolites are so constructed as to exclude dust or damp as completely as possible, the overall dimensions of the instrument are as small as possible, a centring device is incorporated above the levelling base so that the theodolite can be set up under roof station without having to relevel after accurate centring, and it can be manipulated with the minimum amount of movement.

Question 3. What are the functions of a theodolite ?

Answer : The funtions of a theodolite are to measure (a) the horizontal angle, (b) vertical angle, (c) deflection angle, (d) magnetic bearing, and (e) horizontal distancees by tacheometric methods.

Question 4. What is shifting head ?

Answer : The shifting head is a centring device placed below the lower plate but above the tribrach so that it permits the slight movement of isntrument in the horizontal plane without disturbing the level and tripod. It is also known as quick centring device.

Question 5. What is a 115mm theodolite ?

Answer : A theodolite whose horizontal circle has a 115mm diameter is known as 115mm theodolite.

Question 6. What is transverse axis ?

Answer : The transverse axis is the axis about which the telescope and the vertical circle rotate in a vertical plane. It is also known as horizontal or trunion axis.

Question 7. What is vertical axis of the theodolite ?

Answer : The vertical axis is the axis about which the instrument can be rotated in a vertical plane. This is the axis about which the lower plates rotate.

Question 8. What are upper and lower plates of the theodolite ?

Answer : The upper plate of the theodolite, also known as vernier plate, supports the standard at its upper surface and carries two verniers marked A and B with maggnifiers placed at two extremities diametrically opposite (at 180^0). The lower plate, also known as the main scale plate, is circular and have outer edge bevelled. Its bevelled edge is graduated from 0^0 360^0 in clockwise direction.

Question 9. What is the function of clip screw ?

Answer : The verticle leg of the index bar is provided with a clip screw at the lower end. The index bar can be roated slighly with the help of clip screw and the altitude bubble centred for permanent adjustments, if required.

Question 10. What is striding level ?

Answer: A very sensitive level is sometimes mounted at right angles to the telescope axis. It is used in astronomical abservation for levelling the horizontal axis or measuring any error in the level of the axis.

Question 11. What is plunging the telescope ?

Answer : Plunging is the process of rotating the telescope over the horizontal axis through 180^{o} in the vertical plane. After plunging the telescope, the directions of the objective end and eye-piece end is reversed, and the telescope points exactly in the opposite direction.It is also known as transiting or reversing.

Question 12. What is meant by swinging the telescope ?

Answer : Turning of telescope in a horizontal plane is called swinging of the telescope. The swing is termed right or left according to the rotation of the telescope clockwise.

Question 13. What is reverse condition ?

Answer : If the verticle circle is on the right side of the observer, it is called face right or reverse condition.

Question14. What is changing the face ?

Answer : Changing the face is the operation of bringing the telescope from the face left condition to face right observation and vice versa.

Question 15. What are the fundamental lines of a theodolite ?

Answer : There are five fundameental lines or axes of a theodolite. These are :
(a) Verticle axis, (b) Horizontal axis, (c) Line of collimation, (d) altitude bubble axis and (e) Plate level axis.

Question 16. What are the different types of theodolite adjustment ?

Answer : There are two types of adjustments of a theodolite : (1) temporary adjustments which are required to be made at every set-up of the instrument, and (2) permanent adjustments which establish the fixed relationship between the fundamental axes of the theodolite and are essential for the accuracy of the results.

Question 17. What are the methods of measuring horizontal angles in the horizontal plane.

Answer : Depending upon the accuracy and nature of work there are three methods of measuring horizontal angle in the horizontal plane, namely- (1) Ordinary method (2) repetition method, and (3) reiteration method

Question 18. What is reiteration method ?

Answer : The reiteration method, also known as direction method or method of series, is a method of measuring several angles successively from a common vertex and finally closing the horizon.

Question 19. How can a theodolite be used as a level?

Answer: The altitude bubble is first perfectally levelled. Then the zero of the vertical circle (which is fixed to the telescope) is set at the zero of the vernier scale and the telescope is clamped. Under this condition the theodolite can be used as a level.

Question 20. Why two vernier readings are taken ?

Answer : Both vernier readings are taken to eliminate the error due to eccentricity of inner and auter axes.

Question 21. What are the instrumental imperfections of a theodolite ?

Answer : Some defects are non-adjustable, these may be described as instrumental imperfections. Such instrumental imperfections can be allowed for, and they include (i) Eccentricity of verniers, (ii) Eccentricity of centres and (iii) Graduation errors.

Question 22. What do the terms 'temporary adjustments' and 'permanent adjustments' of a theodolite mean ?

Answer : Temporary adjustments are those required everytime instrumeent is set over or under a station.Permanent adjustments are those which are made by the maker, and only tested and corrected occasionally by the observer at comparatively rare intervals when he has reason to think that the instrument has been shaken by some means, and adjustments are out.

Question 23. Give a list of the permanent adjstments of a transit theodolite

Answer : Permanent adjustments of a transit theodolite consists of

1. Adjustment of the plate level.
2. Collimation adjustment.
3. Horizontal axis adjustment.
4. Telescope level adjustment.
5. Verticle circle or index adjustments.

Question 24. What are the defects of simple lenses ?

Answer : The defects of simple lenses include : chromatic and spherical observation, coma, astigmatism, curvature of field and distortion.

Question 25. What is coma ?

Answer : This is one type of oblique spherical aberration of a simple lens and concerns rays not parallel to the principal axis.

Question26. What is magnification ?

Answer. Magnification is the ratio of the focal length of the objective to that of the eye-piece.

Question27. What is "balancing " of the traverse ?

Answer : Balancing is the term applied to operation of applying corrections to co-ordinates so that $\sum L = \sum D$ when the survey forms a closed traverse.

Question 28. What are the common methods of balancing a traverse ?

Answer: The common methods of balancing or adjusting a traverse are
(a) Bowditch's method (b) Transit method, (c) Graphical method, (d) Axis method.

Question 29. What is optical plummet ?

Answer : An optical plummet is an optical centring device which is for accurately centring a theodolite over or under a survey station where centring by plumb-bob is difficult in windy weather or in mine roadways where the velocity of air current is high.

Question 30. What are the linear errors of plumb-bob centreing and plummet centring ?

Answer : Extensive studies have shown the following values of linear errors :

Plumb-bob centring accuracy - 1.2 to 1.5 mm.

Optical plumment accuracy - 0.8 to 1.0 mm.

Question 31. What are the effects of index error or vertical circle and horizontal circle ?

Answer : If the vertical circle verniers do not zero when the line of sight is horizontal, the vertical angles are incorrect.

The index error has no effect on horizontal circle reading.

Question 32. What is Gyro-theodolite ?

Answer : Gyro-theodolite is a Gyro-attachment mounted on a theodolite for determining true meridian at any place during day or night with a high degree of accuracy (± 20") in less than 20 minutes.

Question 33. What is a Repeating theodolite ?

Answer : The repeating theodolite is used for measuring angles by repeatation method. This type of theodolite is provided with double vertical axis (two centres and two clamps) and two or more verniers reading to 10 to 5 seconds.

Question 34. What is direction theodolite ?

Answer : The direction theodolite is an instrument which has only one vertical axis and is provided with two or three micrometer microscope, instead of verniers, to read fraction parts directly to nearest second and 0.1 sec. by estimation.

Question 35. What are the permissible errors of underground traverse which has been colsed polygonally ?

Answer : The permissible arrors are :

(a) Angualr error (Total) : $(20+x)\sqrt{n}$ seconds ; where n = observing stns.
and x = 10 secs. or smallest reading in second of the vernier (least count)

(b) Error of closure :The ratio of closing error (as found out from computation of lats and dips) and sum of hor. length of draft shd. not exceed 1:1500.

Question 36. What is 'residual error' ?

Answer : The adjustment of theodolite is being very delicate, perfect adjusment is not possible and consequently, very small errors remain unadjusted which are residual error.

Question 37. What is levelling head ?

Answer : The levelling head consists of two parallel plates, the upper plate is tribrach and the lower one is trivet stage or foot plate. These two plates are connected through the levelling screws of the instrument.

Question 38. What is the effect of wind in theodolite surveying ?

Answer : Strong winds cause vibration in the instrument and it becomes difficult to work. They also deflect the plumb bob string out of plumb, and the centring becomes difficult.

Question39. What is lining-in ?

Answer. The establishment of various points on a given line is known as lining-in.

Question40. What is double sighting ?

Answer : For prolongation of survey line with high precision, especially when the instrument is in poor adjustment, double reversing from the instrument station is used which is known as double sighting

CHOOSE THE CORRECT ANSWER

1. The theodolite is which the telescope can not be revolved through complete revolution about its horizontal axis in the vertical plane is called ?
(a) transit theodolite (b) non-transit theodolite (c) microptic theodolite.
2. The size of a theodolite is defined according to
(a) length of telescope (b) diameter of object glass
(c) diameter of graduated horizontal circle.
3. The trunnion axis is also known as
(a) transverse axis (b) vertical axis (c) telescope axis.
4. The process of turning the telescope about the vertical axis of the instrument in the horizontal plane is
(a) plunging (b) reversing (c) swinging.
5. The face left position is laso called
(a)Telescope inverted (b) telescope normal (c) telescope reversed
6. Fine adjustment in a thedolite is done by
(a) focusing screw (b) tangent screw (c) clamp screw.
7. For improved accuracy the included angle is measured by
(a) repetition method (b) reiterration method (c) deflection angle method.
8. When the theodolite is in perfect adjustment, the axis of the plate levels must be
(a) prependicuular to vertical axis (b) at right angle to the horizontal axis
(c) prepeendicular to trunnion axis.
9. The upper plate of a theodolite is fixed to
(a) outer spindle (b) inner spindle (c) levelling head.
10. Turning the telescope in the vertical plane about the horizontal axis is called
(a) transiting (b) lining-in (c) swinging
11. For measuring horizontal angle between two lines, with better accuracy than that given by the vernier, the method is commonly used is
(a) reiteration (b) repetition (c) double sigting.
12. The final setting of the plates when taking a foresight is achieved by using the
(a) upper clamp screw (b) lower clamp screw (c) upper tangent screw
(d) lower tangent screw .

13. Balancing of traverse is done accrding to
(a) transit rule (b) Cosine rule (c) Weddle's rule

14. The erroe which can not be eliminated be averaging face right and face left obervation is
(a) collimation error (b) imperfect adjustments of plate level
(c) error due to horizontal axis not being perpendicular to vertical axis.

15. The error due to imperfect graduations on the hirizontal scale can be minimised by
(a) applying index correction (b) double centring
(c) taking observations over different portions of the scale.

16. In a theodolite traverse for land surveying the length of the traverse lines are generally measured with a
(a) chain (b) metallic tape (c) steel tape (or band) (d) invar tape

17. The most modern theodolite for measuring angles precisely is of
(a) double reading type (b) direct scale reading type (c) repeating type

18. A Gyro is an attachement mounted ion a theodolite for determining
(a) true meridian (b) magnetic meridian (c) artifical meridian

19. The absence of chromatic aberration is
(a) aplanatism (b) achromatism (c) astigmatism

20. Aplanatism means the absence of
(a) chromatic aberration (b) spherical aberration (c) distortion.

21. The eye-piece used in a telescope for astronomical observations or highly inclined shafts, winzes or raised is
(a) Ramsden eye-piece (b) diagonal eye-piece (c) erecting type eye-piece.

22. When the auxiliary telescope is attached to the main telescope, it is known as a
(a) top telescope (b) bottom telescope (c) side telescope

23. When the auxiliary telescope is attached to the transit axis it is known as
(a) top telescope (b) bottom telecope (c) side telescope

24. The line of sight must be
(a) coincided with optical axis (b) parallel to trunnion axis
(c) at right angles to the axis of telescope

25. Of the five permanent adjustments of a transit theodolite
(a) two adjustments are required for horizontal angles.
(b) three adjustments are required for vertical angles.
(c) two adjustments are required for horizontal angles

26. The reading of the verniers on the vertical circle should be zero when
(a) the line of sight is horizontal (b) the line of sight is vertical
(c) the line of collimation is parallel to vertical axis

27. The tribrach and the trivet stage are connected through the
(a) standards (b) levelling screws (c) inner spindle

28. A repeating theodolite has a
(a) single vertcal axis (b) Double vertical axis (c) triple vertical axis.

29. Wild T-2, T-3 and T-4 theodolites are
(a) Repeating line (b) Direction type (c) Direct scale reading type.

30. Watts microptic theodolite No. 1 fall under category
(a) Repeating type (b) Direction type (c) direct scale reading type

31. Poor focusing is a source of
(a) instrumental error (b) personal error (c) natural error.

32. The most common method of traversing with a theodolite is
(a) continuous azimuth method (b) double foresight method
(c) deflection angle method

33. If the reading of the verniers on the vertical circle is not zero when the line of the sight is horizontal the error is known as
(a) residual error (b) index error (c) graduation error.

34. Acromatism means the absence of
(a) spherical aberration (b) chromatic aberration
(c) coma (d) distortion

35. The levelling head consists of
(a) two standards (b) two stages (c) two circular plates.

36. The shifting head of a theodolite is a
(a) centring device (b) tripod (c) finder collimator device.

37. At the two ends of the index arm of a transit theodolite are fitted the
(a) altitude bubble (b) clip screw (c) verniers

38. The vertical leg of the T-frame of theodolite is provided with a frok and
(a) Verniers C and D (b) clip screw (c) altitude bubble.

39. The lower plate of a theodolite is known as
(a) horizontal circle (b) main scale plate (c) both a. & b.

40. A very sensitive level mounted at right angles to the telescope axis is known as
(a) altitude bubble (b) striding level (c) plate level

41. The vernier clamp of a theodolite is
(a) top clamp (b) bottom clamp (c) body clamp

42. The ratio of the focal length of the objective to that of the eye - piece is known as
(a) size of field (b) magnification (c) definition

43. The upper plate of a theodolite to the
(a) inner spindle (b) outer spindle (c) hollow tapered sindle

44. The error due to defective centring varies inversely as the
(a) length of telescope (b) height of instrument (c) length of sight.

45. The latitude of a line is obtained by multiplying its length by
(a) tan θ (b) sin θ (c) cos θ (d) cot θ
where θ is the reduced bearing.

46. If angural measuremtns of a traverse are more accurate than the linear measurements, balancing of the traverse is generally done by
(a) Bowditch's rule (b) Transit rule (c) Axis method

47. The most accurate method of plotting a theodolite traverse is
(a) consecutive co-ordiantes (b) independent co-ordinates (c) protractor

48. The omitted measurements can be computed if they are not more than
(a) 1 (b) 2 (c) 3 (d) 4

49. The bowditch's rule is based on the assumption that the probale errer is proportional to
(a) L (b) $\sqrt{L}$ (c) L^2
where L is the length of the line.

50. The sum of the interior angles of a closed theodolite traverse of n side is euqal to
(a) $(2N - 4) \times 180^0$ (b) $(N - 2) \times 180^0$ (c) $(N - 4) \times 180^0$

Answer : 1 (b), 2 (c), 3 (a), 4 (c), 5 (b), 6 (b), 7 (a), 8 (a), 9 (b), 10 (a), 11 (b), 12 (c), 13 (a), 14 (b), 15 (c), 16 (c), 17 (b), 18 (a), 19 (b), 20 (b), 21 (b), 22 (a), 23 (c), 24 (a), 25 (c), 26 (a), 27 (b), 28 (b), 29 (b), 30 (a), 31 (b), 32 (b), 33 (b), 34 (b) 35 (b), 36 (a), 37 (c), 38 (b), 39 (c), 40 (b), 41 (a), 42 (b), 43 (a), 44 (c), 45 (c), 46 (b), 47 (b) 48 (b), 49 (b), 50 (a).

SETTING OUT CURVES

SHORT QUESTIONS

Question 1. What is a curve ?

Answer : Gradual and smooth change of direction from one straight to another is known as curve.

Question 2. What are the different types of horizontal circular curves ?

Answer. The horizontal curves are (a) simple curve, (b) compound curve, (c) reverse curve,and (d) transition curve.

Question 3. What is compound curve ?

Answer. A compound curve is one in which a curve of one radius runs into another of different radius bending in the same direction with a common tangent and with their centres on the same side.

Question 4. What is a surpentine curve ?

Answer. The surpentine curve is reverse curve which is composed of two arcs of equal or different radius bending in opposite direction with a common tangent and with their centres on opposite sides of the curve.

Question 5. What is the degree of a curve ?

Answer . According to the arc definition, the degree of curve is defined as the central angle of the curve that is subtended by an arc of 30m. length. According to the chord definition, it is defined as the central angle of the curve that is subtended by its 30m. chord length.

Question 6. What is the degree of curve in metric system. ?

Answer. Degree of curve in metric system is angle at thee centre subtended by an arc (or chord) of 20 metres and of 10 metres.

Question 7. What is the purpose of transition curve ?

Answer. The purpose of transittion curve is to avoid inconvenience to the passengers, sitting in a vehicle, due to the shock experienced by them when a vehicle suddenly enters on a circular curve from a straight.

Question 8. What are the forces acting when a vehicle moves on a curve ?

Answer. Two forces are (i) weight of the vehicle and (ii) the centrifugal force.

Question 9. What is the purpose of super elevation ?

Answer. The purposes are (a) to prevent excessive wear on the outer wheel, (b) to equalise the pressure on the two wheels, and (c) to prevent the vehicle from over turning.

Question 10. What are the methods for calculating the length of the transition curve. ?

Answer. There are three methods for calculating the length of the transition curve : (i) By an arbitrary gradient, (ii) By the time rate, and (iii) By the rate of change of radial acceleration.

Question 11. What is centrifugal ratio ?

Answer. The ratio of centrifugal force and the weight is called centrifugal ratio.

Question 12. What is the radius of 1^0 curve ?

Answer. The radius of a 1^0 curve is 1719m.

Question 13. What is the versed sine of a curve ?

Answer. The distance between the apex of a curve and the mid-point of a long chord is known as versed sine or rise or middle ordinate of the curve.

Question 14. What is 4^0 curve ?

Answer. When the unit chord (30m) subtends an angle of 4^0 at the centre, the corresponding curve is known as 4^0 curve.

Question 15. What is the relation between the radius and degree of a curve?

Answer. Radius = 1719/D where D = degree of curve.

Question 16. Two parallel lines are to be connected. What type of curve would you suggest ?

Answer : A reverse curve should be adopted.

Question 17. What are the different types of vertical curves ?

Answer : Vertical curves are of two types. Summit curves and sag curves.
Depending upon the magnitude and sign of the gradients at the intersection point they are: Summit curves - (i) an upgrade followed by a downgrade, (ii) a down grade followed by another down grade, and sag curves, (iii) a downgrade followed by an upgrade.(iv) an upgrade followed by another upgrade.

Question 18. How is the gradient or grade of a rail-road or a highway expressed ?

Answer. Gradient may be expressed in two ways- (a) by percentage : the vertical rise (or fall) in a horizontal distance of 100m (b) by ratio : ratio of vertical rise of fall and the horizontal distance.

Question 19. On what basis is a vertical curve designed?

Answer. A vertical curve is designed on the basis of sight distance.

Question 20. What is the type of vertical curve preferred ?

Answer. A parabolic curve is preferred, because it is easy for a vehicle to pass over and it is simple to provide minimum sight distance.

Question 21. What is the tangent correction ?

Answer . While constructing a vertical curve, cutting or filling has to be done to obtain the desired shape. The height of cutting or filling is known as tangent correction ?

Question 22. What is sight distance ?

Answer. The sight distance is the length of the road ahead which is visible to the driver of a vehicle.

Question 23. What is cant ?

Answer. If the radius of the curve is small, speed of the vehicle high and track laid horizontal, there would be danger of the vehicle become derailed. To avoid such danger the outer rail is raised to a higher level than the inner rail and the amount by which it is raised is called cant or super elevation.

Question 24. What are the methods of setting out a circular curve can be depending upon the equipment used ?

Answer. The methods of setting out a circular curve can be classified into five types : (i) Tape methods, (2) Tape and theodolite methods, (3) Ttwo theodolites method, (4) Tacheometric method, and (5) Total station method.

CHOOSE THE CORRECT ANSWER

1. When a curve of one radius runs into another of different radius bending in the same direction with a common tangent and with their centres on the same side, it is a
   (a) Simple curve (b) Compound curve (c) Surpentine curve.
2. Reverse curve is also known as
   (a) transition curve (b) deviation curve (c) ogee
3. A non - circular curve, which is introduced between a straight and a circular curve or between two branches of a compound or reverse curve, is
   (a) transition curve (b) deviation curve (c) surpentine curve
4. A compound curve is one that is composed of two or more arcs of
   (a) different tangent (b) different radii (c) bending in different direction
5. The distance from the centre of the long chorrd to the centre of the curve and measured at right angles to the long chord is
   (a) apex distance (b) tangent distance (c) versed sine of the curve
6. The versed sine of a curve is
   (a) Middle ordinate (b) Rise (c) both (a) and (b)
7. In order to avoid derailment of a vehicle moving on a rail curve, the outer rail is raised to higher level than the inner rail and the amount of which is called
   (a) ogge (b) cant (c) rise
8. The type of curve used for shunting of railway in parallel lines is
   (a) transition curve (b) reverse curve (c) deviation curve
9. A curve which has a large number of centres on the same side is known as
   (a) compound curve (b) reverse curve (c) surpentine curve
10. A transition curve is also known as
    (a) sag curve (b) spiral curve (c) deviation curve
11. A curve is designated on the basis of
    (a) length (b) radius (c) long chord

12. A vertical curve is designatad on the basis of
(a) change of gradient (b) radius of the curved (c) minimum sight distance

13. A vertical curve is considered as a/ an
(a) parabolic curve (b) elliptical curve (c) deviation curve

14. The grade of a rail road or highway is expressed in
(a) percentage (b) ratio (c) both (a) and (b)

15. The radius of 1^0 curve is
(a) 1719 m (b) 1760 m (c) 2000 m

16. The first tangent is also known as the
(a) forward tangent (b) back tangent (c) apex tangent

17. When two tangents AB and BC meet at B, the point B is known as the
(a) summit (b) apex (c) vertex

18. A simple curve is designated by
(a) degree of curve (b) radius of curve (c) both (a) & (b)

19. The necessity of providing a simple curve on a road is to change the
(a) grade (b) direction (c) both (a) & (b)

20. The distance of the point of intersection to the mid-point of the curve is
(a) rise (b) apex distance (C) versed sine of the curve

21. The length of long chord and the tangent of a circular curve of radius R will be equal if the angle of deflection is
(a) 30^0 (b) 60^0 (c) 120^0

22. if the degree of curve (specified length 30 m) is 3^0, the radius of curve is approximately
(a) 573 m (b) 1910 m (c) 380 m

23. The vertical curve having convexity upwards is called
(a) Valley curve (b) Summit curve (c) sag curve

24. Super-elevation in cms is
(a) track gauge in cms x $\frac{V^2}{gR}$ (b) track gauge in cms x $\frac{V}{gR}$
(c) track gauge in cms x $\frac{V^3}{gR^2}$

25. For a design speed of 100 km per hour, the sight distance on the summit of a vertical curve is approximately
(a) 300 m (b) 150 (c) 75 m

26. The vertical curve having concavity upwards is known as
(a) crest curve (b) summit curve (c) valley curve

27. The method which is more accurate for laying out important curves belowground is
(a) chords and offset method (b) chords and angle method
(c) two theodolites method

28. If the 0 is the angle subtended by the chord at the centre of a curve, and R is the radius of the curve, the lentgh of chord =
(a) R x tan $\frac{0}{2}$ (b) $2\pi R \times \frac{0}{360}$ (c) 2R Sin $\frac{0}{2}$

29. If the radius and the deflection angle are known, tangent distance =
(a) R Sin $\frac{\theta}{2}$ (b) R sec $\frac{\theta}{2}$ (c) R tan $\frac{\theta}{2}$

30. The allowable centrifugal ratio (CR) for railway is
(a) $\frac{1}{4}$ (b) $\frac{1}{6}$ (c) $\frac{1}{8}$

31. The first tangent point also known as
(a) point of start (b) point of tangent (c)point of curve

32. The second tangent point is also known as
(a) end point (b) point of tangency (c) rear point

33. The versed sine of a curve is given by
(a) R (1 - cos $\frac{\theta}{2}$) (b) R (sec $\frac{\theta}{2}$ - 1) (c) R (1 - tan $\frac{\theta}{2}$)

34. The apex distance is given by
(a) R (1 - sin $\frac{\theta}{2}$) (b) R (cos $\frac{0}{2}$ - 1) (c) R (sec $\frac{\theta}{2}$ - 1)

35. The length of a unit chord should not be more than
(a) $\frac{1}{10th}$ of the radius (b) $\frac{1}{20th}$ of the radius (c) $\frac{1}{50th}$ of the radius

36. An ideal transition curve is also known as
(a) clothoid curve (b) cubical curve (c) parabolic curve

37. In two-theodolite mehod of setting out of curve requires
(a) Two theodolites, chain and tape (b) two theodolites and chain or tape
(c) only two theodolites

38. The super elevation is known as
(a) easement curve (b) cant (c) ogee

Answer : 1 (b), 2 (c), 3 (a), 4 (b), 5 (c), 6 (c), 7 (b), 8 (b), 9 (a), 10 (b), 11 (b), 12 (c), 13 (a) 14 (c), 15 (a), 16 (b), 17 (c), 18 (c), 19 (b), 20 (b), 21 (c), 22 (a), 23 (b), 24 (a), 25 (b), 26 (c), 27 (b), 28 (c), 29 (c), 30 (c), 31 (c), 32 (b), 33 (a), 34 (c), 35 (b), 36 (a) 37 (c), 38 (b).

TACHEOMETRIC SURVEYING

SHORT QUESTIONS

Question 1. What is tacheometry ?

Answer : Tacheometry is a branch of angular surveying in which the horizontal and vertical distances are measured by instrumental observations without any linear measurements.

Question 2. What is the other name of tacheometry ?

Answer . Tacheometry is also known as Telemetry.

Question 3. What is the difference between a tacheometer and a theodolite ?

Answer. When a theodolite is fitted with anallatic lens, it is known as tacheometer. A theodolite without the anallatic lens is a simple transit theodolite.

Question 4. Where is the tacheometry applicable ?

Answer. As a means of surveying, tacheometry is applicable in rough and difficult country where ordinary levelling is tedious, and chaining inaccurate, difficult and slow.

Question 5. What is stadia diaphragm ?

Answer. The diaphragm of the telescope fitted with two additional horizontal cross-hairs one above and the other an equal distance below the middle horizontal cross-hairs one above and the other an equal distance below the middle horizontal cross-hair is known as stadia diaphragm.

Question 6. Why is an anallatic lens used in a tacheometer ?

Answer : The anallatic lens, an additional convex lens, is fitted in a tacheometer to eliminate the additive constant.

Question 7. Why anallatic lens is never fitted to internal focussing telescope ?

Answer : Introduction of anallatic lens in a telescope increases the absorption of light within the telescope thereby causing loss of brilliance in the image. It reduces the maximum distance of which the staff can be accurately read. Anallatic lens is not required in internal focussing telescope , but by the choice of suitable optical characteristics it has been found possible to construct the telescope in such a way that the additive constant is so small as to be negligible.

Question 8. What is the difference between a fixed hair and movable hair methods ?

Answer : In fixed hair method, the vertical spacing between the upper stadia hair and lower stadia hair, called stadia interval, is fixed. This spacing is not changed during the measurement. In movable hair method, the vertical spacing between the stadia hairs (i.e. stadia interval) can be varied by moving that stadia hairs vertically by micrometer screws. In movable hair method, the staff intercept is fixed.

Question 9. What is a subtense bar ?

Answer : The subtense bar is a horizontal staff 4m long consisting of a small spirit level and two circular discs at its two ends for measuring the horizontal angle between targets, from which the horizontal distance between the instrument station and the bar may be calculated using the fixed horizontal length between two discs and the subtended angle on instrument.

Question 10. What are the multiplying and additive constants of a tacheometer ?

Answer : In tacheometry, the horizontal distance D from the vertical axis of the tacheometer to the staff station is given by

$D = \frac{f}{i} s + (f + d)$ where d is the distance of the vertical axis of the instrument from the optical centre.

The quantity $\frac{f}{i}$ is known as multiplying constant or stadia interval factor, and has a value of 100. The quantity (f + d) is known as additive constant.

Question 11. What is stadia traverse ?

Answer : The stadia traverse is similar to theodolite traverse, but in this case the traverse lines are measured indirectly by tacheometric method and not directly with a tape.

Question 12. What is tangential tacheometry ?

Answer : In this system of tacheometry, the angles of elevation or depression are measured and the distances are calculated from the values of tangents of the angles.

Question 13. What are stadia diagrams ?

Answer : Stadia diagrams or stadia charts are tables which are used for reduction of stadia notes, to determine the horizontal distance and the vertical intercept graphically.

Question 14. What are stadia slide rule ?

Answer : The stadia slide rule is a mechanical device in the form a ruler about 250mm long. It is used to reduce the vertical angles in to the horizontal distance and the vertical intercept.

Question 15. What is the principle of tacheometry ?

Answer : The principle of tachemetry is based on the property of isosceles triangles where the distances of the bases from the apices and the length of the bases always bear a constant ratio.

Question 16. What are the basic systems of tacheometric measurements ?

Answer : There are basically three systems of tacheometric measurements : 1. Stadia systems (a) Fixed hair method, (b) Movable hair method. 2. Tangential system, 3. Subtense bar system.

Question 17. What are the methods to determine the multiplying constant and additive constant ?

Answer : Two methods are commonly used.

(i) In this method, only multiplying constant 'K' is determined from the field observations and the additive constant C is determined by direct measurement of f and d.

(ii) In this method, both the constants are determined from the field observations.

Question 18. What is anallatic lense ?

Answer : The anallatic lense is an extra convex lens, placed between eye-piece and objective at a fixed distance from the objective glass. The fitting of this extra lens eliminates the need of an additive constant.

Question 19. What does 'reduction of readings' mean ?

Answern : The calculation of horizontal and vertical distances by the direct application of formulae every time is very laborious. So, to simplify the calculation, some tables or graphs are prepared which are known as reduction of readings.

Question 20. What is the purpose of direct readings tacheometer ?

Answer : Direct readings tacheometers are designed to obtain horizontal and vertical distances directly from staff readings without any need for labourious calculations or computation work.

Question 21. What are the permissible closing errors in a closed traverse ?

Answer. (a) The linear error of closer in a stadia traverse should not exceed $0.005\sqrt{P}$ metres, where P is the perimeter of the traverse in metres.
(b) The error of closure in elevation may vary from $0.08\sqrt{M}$ to $0.25\sqrt{M}$ metres, where M is the distance in kilometres.

Question 22. What are the specified standards of precision in a single horizontal distance and in a single vertical distance.

Answer. The error in a single horizontal distance should not exceed 1 in 500 and in a single vertical distance should not be more than 0.1m.

CHOOSE THE CORRECT ANSWER

1. A transit theodolite fitted with a stadia diaphragm and anallatic lens is known as /an
   (a) Astronomical theodolite (b) Telemeter (c) Subtence theodolite.
2. the tacheometry is also known as
   (a) Tacymetry (b) Telemeter (c) both (a) & (b) (d) Nither (a) nor (b)
3. A tacheometer differs from an ordinary theodoite in one respect that its diaphragm is fitted with
   (a) Two additional vertical hairs (b) Two additional horizontal hairs
   (c) Three additional horizontal hairs (d) Three additinal vertical hairs
4. The stadia diaphragm is provided for measuring
   (a) Elevation (b) Bearing (c) Horizontal distance
5. The method of tacheometry commonly used in practice is
   (a) Fixed stadia system (b) Movable stadia system
   (c) Tangential system (d) Subtense bar system
6. The observation to the staff points are known as
   (a) Side shots (b) Ranging (c) Auto reduction
7. Levelling can also be performed with a tacheometer
   (a) Directly and Rapidly (b) Indirectly and rapidly (c) Slowly and indirectly

8. Stadia diagrams are also called
   (a) Reduction diagrams (b) Stadia tables (c) Stadia circle diagram.
9. For long distances a special large staff used in tacheometry is called
   (a) Levelling staff (b) Stadia rods (c) Beaman staff
10. The stadia slide rule is a machanical device in the form of ruler about
    (a) 100 mm long (b) 250 mm long (c) 150 mm long
11. In a self-reducing tacheometer, the stadia hairs of the diaphragm are repllaced by
    (a) Three horizontal lines (b) Three vertical lines (c) Three curved lines.
12. The tangential method of tacheometry is generally used when the diaphragm having
    (a) Two additional horizontal hairs (b) Two additional hairs (c) No stadia hairs
13. The object of providing the annallatic lens in the telescope is to make the additive constant (f + d) exactly
    (a) 100 (b) zero (c) Neither (a) nor (b)
14. The multiplying constant is donated by
    (a) $\frac{f}{d}$ (b) f + d (c) $\frac{f}{i}$ s
15. An anallatic lens is not required with and is never fitted to
    (a) Internal focusing telescope (b) Exteranal focusing telescope (c) Neither (a) nor (b)
16. The movable hair method of tacheometry is also known as
    (a) Self-reducing method (b) Subtense method (c) Taangential method
17. Subtense bar system is generally used for measurement of horizontal distance in
    (a) undulating areas (b) Mountainous areas (c) Flat areas
18. The stadia markings are made on
    (a) Eye piece (b) Diaphragm (c) Levelling staff
19. The usual value of multiplying constant is
    (a) 100 (b) 150 (c) 200
20. Stadia rod is used for
    (a) Long sights (b) short sights (c) Both (a) & (b)
21. The error in a single horizontal distance should not exceed in tacheometry
    (a) 1 in 100 (2) 1 in 500 (c) 1 in 1000
22. The subtense bar is used to measure
    (a) Vertical distance (b) Horizontal distance (c) Elevation
23. As the distance between the tacheometer and staff increases, the staff intercept by stadia hair
    (a) Increases (b) Decreases (c) Remain constant
24. In tangential tacheometry the staff is held
    (a) Inclined (b) Normal to line of sight (c) vertically

25. When the line of sight inclined and the staff is held vertically the horizontal distance is given by

(a) $\frac{f}{i} \times S \cos^2 \theta + (f + d) \cos \theta$

(b) $\frac{f}{i} \times S \sin^2 0 + (f + d) \sin 0$

(c) $\frac{f}{i} \times S \cot^2 \theta + (f + d) \cot 0$

Answer. 1 (b) 2 (c), 3(b), 4(c), 5(a), 6(a), 7(b), 8(a), 9(b), 10(b), 11(c), 12(c), 13(b), 14(b), 15(a),16(b), 17(a), 18(b), 19(a), 20(a), 21(b), 22(b), 23(a), 24(c), 25(a).

TRIANGULATION, TRILATERATION AND TRAVERSING

SHORT QUESTIONS

Question 1. What are the methods used for conducting the geodetic surveys ?

Answer : There are mainly three methods for conducting geodetic surveys. They are : Triangulation and traversing.

Question 2. What is the main principle of triangulation ?

Answer . The main principle of triangulation is based on the trigonometrical proposition that if only one side, called the base line, is measured with considerable care and accuracy and all the angles of the network of interlacing triangles are very carefully observed then the remaining sides can be calculated by the sine rule.

Question 3 . Why horizontal control is required in surveying ?

Answer . A horizontal control survey is reuired to estaablish the positions in plan of a number of control points on or near the surface of earth. The control points so determined furnish the most precise control to which a more detailed survey of intervening country may be referred.

Question 4. What is the basis of classification of triangulation figure ?

Answer. The basis of classification is the accuracy with which the length and azimuth of a line of the triangulation are determined.

Question 5. What are the accepted grades of triangulation ?

Answer. The accepted grades of triangulation are (a) First order or primary triangulation , (b) Second order or secondry triangulation, and (c) Third order or tertiary triangulation.

Question 6. What are the commonly used arrangements of triangles in a triangulation system.

Answer. The common figures in triangulation are (a) single chain of tringles, (b)double chain of triangles, (c) Central point figures, (d) Quadrilateral, (e) A combination of the above systems.

Question 7. What is the meant by well-conditioned triangles ?

Answer. A triangle is called well-conditioned if its shape is such that any error in the measurement of an angle has a minimum effect upon the computed lengths. The best shape of a triangle is an isoscelles triangle whole base angles are 54° 14'. It has been found that an equilateral triangle is the most suitable and may be taken as the most wel-conditioned triangle. In actual practice, the triangle having an angle less than 30° or more than 120° should be avoided.

Question 8. What is reconnaissance for a triangulation ?

Answer. Reconnaissance is the preliminary field inspection of the entire area to be covered by triangulation and collection of relevant data.

Question 9. What is the purpose of signals in triangulation ?

Answer . To define exact position of a triangulation station so that it can be observed from other station, signals are erected vertically over the station marks.

Question 10. What is the classification of signals ?

Answer. The signal may be classified as (a) Daylight or non-luminous (opaque) signal, (b) sun or luminous signal, and (c) Night signal.

Question 11. What is a tower ?

Answer. A tower is a structure erected over a station for the support of the instrument and the observer and is provided when the station or signal, or both are to be elevated.

Question 12. What is heliotrope ?

Answer. Heliotropes or sun signals are luminous signals which reflect the rays of the sun towards the instrument station. A heliotrope consists of a circular plane mirror and some device for pointing the mirror so that the sunlight reflected by the mirror to be seen at the instrument station.

Question 13. What is phase of signal ?

Answer . Phase of signal is the error of bisection which arises from the fact that, under lateral illumination, the signal is partly in light and partly in shade. As a result the signal is not bisected at the centre and an error of bisection is introduced.

Question 14. What is base line ?

Answer. The line on which the framework of the triangulation survey is built is known as base line. If the angles of the triangles in triangulation are very carefully observed and at least one of the sides or base line is measured with care and accuracy, the lengths of all the remaining sides can be obtained by sine rule.

Question 15. What are the methods of base line measurement ?

Answer. The methods of the measurement of the base line may be classifed as

1. Tape in catenary and tape along the grround methods.
2. Tacheometric method.
3. E.D.M. instruments method.

Question 16. What should be the accuracy in colliery triangulation ?

Answer. In colliery triangulation an accuracy of 1 in 5000 for calculated distances between two stations should be attained. For this accuracy, the triangle closure should not exceed ±15 seconds which is the maximum permissible error.

Question 17. Under what conditions extensions of base line become necessary ?

Answer : The measurement of a long base-line may sometimes be difficult due to obstacles or in hilly areas. I such cases, only a portion of the base-line accurately measured. In order to form well-conditioned triangles, the measured base line is extended in length.

Question 18. What are computations in a plane triangulation ?

Answer : Computation in a plane triangulation system are usually done in the following sequence -- (a) adjustment of observed angles, (b) computation of lengths, (c) computation of azimuths and all lats. and deps., (d) computation of independent co-ordinates.

Question 19. What is the usefulness of Snellius Rhombus in triangulation survey ?

Answer : Measurement of a longer base line by impracticable due to obstacles and the accuracy of linear measurement my be adversely affected by poor tapographical conditions. To solve the problem of obstacles, it is therefore better to measure a short base line very accurately and extended by employing some trigonometrical method. This system of base extension is done by Snellius Rhombus.

Question 20. Why is observation of circumpolar star at equal attitudes preferred in colliery surveying ?

Answer : For finding true meridian or true north, the observation of circumpolar star at equal attitudes is preferred in colliery surveying owing to the method being simple and accurate, and independent of latitude and time.

Question 21. After measuring the horizontal angles in a triangulation survey, what are the adjustments required to determine their most probable values ?

Answer : Two step adjustments are made in order to determine the most probable values :
(1) Station adjustment, and (2) Figure adjustment.

Question 22. What is the principle of trilateration ?

Answer : The principle of trilateration is based on the trigonometrical proposition that if the three sides (laterals) of a triangle are known then all the angles of the triangle can be computed from the trigonometric formulae.

Question 23. What is broken base-line ?

Answer : When the whole length of a base line can not be measured because the intermediate length of it contains obstruction, the base may be lined out and the two lengths on either side of the obstruction is measured in the usual way. To obtain the length of the intermediate section a theodolite is set up at suitable point so that all the four ends of the two lengths on either side of obstruction can be observed and all the angles are measured at the station and finally the required length, known as broken base, is calculated trigonometrically.

Question 24. What is a satellite station in triangulation survey ?

Answer : When a church spires, towers of temples, etc. are selected as triangulation station, the instrument can not be set up over true stations for the measurement of horizontal angles to the triangles. In such cases a subsidiary station, as near as possible, to the true station is selected and the angles are measured to the three triangulation stations. Such subsidiary stations are called satellite stations.

Question 25. What is eccentric distance ?

Answer : The distance between the true station of triangulation and the subsidiary or satellite station is known as eccentric distance.

Question 26. What are the different accurate methods for determining the true meridian at any place ?

Answer : The methods for determining the true meridian at any place are :
(i) Equal altitude of the sun or circumpolar star, (ii) Extra-meridian observation of the sun or

circumpolar star, (iii) Two greatest elongation of a circumpolar star, (iv) Single observation of a circumpolar star at elongation, (v) Gyro-theodolite.

Question 27. What is a Pole star ?

Answer : The Pole star is a bright star which forms the first star in the tail of circumpolar constellation known as Ursa Minor or Little Bear of Little Dipper. The Pole star describes a small circle about the invisible pole once each sidereal day.

Question 28. How will you determine the true north approximately by a watch ?

Answer : A watch is held so that the hour hand points to the sun, and the angle formed between the hour hand and the '12' mark on the dial is bisected. If the bisected line is prolonged backward it will indicate the direction of the true north.

Question 29. What are uses of EDM in modern mine surveying?

Answer: The uses of electronic distance measuring instruments in mine surveying are : (1) Establishment of control points by trilateration,(2) Measuring the base line in triangulation, (3) Rapid establishment of ground control points for photogrammetry,(4) Measurement of traverse lines precisely within a very short period,(5) Measuring the depth of shaft.

Question 30. What is base net?

Answer: The group of triangles meant for extending the base line is known as base net.

CHOOSE THE CORRECT ANSWER

1. Primary triangulation is known as
(a) First order (b) Second order (c) Third order

2. The primary triangulation system embraces the
(a) Small area (b) Medium area (c) Vast area

3. Length of base line in primary triangulation is
(a) 0.5 to 3 km (b) 2 to 5 km (c) 5 to 15 km

4. In G.T.S. (India) ten base -lines were used, the length of nine bases vary from 6.4 to 7.8 miles and that of the tenth base is
(a) 3.5 miles (b) 2.3 miles (c) 1.7 miles

5. All of the ten base-lines of G.T.S. were measured with the Colby apparatus which used
(a) Flexible wire (b) Rigid bar (c) Invar tape

6. The commonly used arrangement of triangles in triangulation is called
(a) Layouts (b) Snellious rhombus (c) Isosceles triangle

7. The triangulation station at which astronomical observations are made for azmuths is called
(a) Grid station (b) Lapelace station (c) Satellite station

8. When the primary triangulation network consists of a series of chain of braced quadrilaterals which usually run in two prependicualr direction, generally east-west and north-south, the framework is known as ?
(a) Central system (b) Grid iron system (c) Intersection system

9. The work of accurate measurement of base line has been considerably simplified and accurate with the advent of
(a) Tacheometry (b) Altimeters (c) EDM instrument

10. Measurement of base line by invar tape over measuring tripods and the ends of the tape is connected to straining tripods is known as
(a) Wheeler's method (b) Jaderin's method (c) Hunter's short base method

11. In rough and undulating grounds or in hilly countries the base line is measured
(a) By tape in catenary (b) by tape along the ground (c) Neither (a) nor (b)

12. The subsidiary station is known as
(a) Satellite station (b) False station (c) Both (a)& (b) (a)Neither (a) nor (b)

13. The most accurate method of fixing the horizontal control points is
(a) Triangulation (b) Trilateration (c) Traversing

14. Satellite station is also known as
(a) Eccnetric station (b) Trigonometric station (c) True station

15. The distance between the true station and the subsidiary station is known as
(a) True distance (b) Reduction distance (c) Eccentric distance

16. The figure most commonly used in triangulation is a
(a) Braced quadrilateral (b) Quadrilateral with a centre point
(c) Quadrilateral with one diagonal

17. In a braced quadrilateral, the position of two unknown corner points can be determined by
(a) Two alternative routes (b)Three alternative routes (c) Four alternative routes

18. A satellite station is required when a main station
(a) Can not be sighted (b) Can not be occupied (c) both (a) & (b)

19. The elevated towers are used for
(a) Signals (b) Intruments (c) both (a) & (b)

20. The group of triangles meant for extending the base line is
(a) Ecentric base (b) Check base (c) Base net

21. The appical angle of the triangles of Snellius Rhombus should not be less than
(a) 20° (b) 30° (c) 60°

22. The observation of circumpolar star at equal altitudes is preferred in colliery triangulation for finding
(a) Magnetic meridian (b) True meridian (c) Longitude

23. For finding true north the instrument used is
(a) Geodimeter (b) Rader apparatus (c) Gyro-theodolite

24. Trilateration is based on the
(a) Trigonometric proposition (b) Base net (C) very large triangle

25. The secondery triangulation consists of a number of points fixed within the nework of
(a) Primary triangulation (b) Tertiary triangulation (c) Both (a) & (b)

26. Degree of accuracy in secondary triangulation is
(a) 1 in 5000 (b) 1 in 50000 (c) 1in 10000

27. Complex figures in triangulation should not involve more than
(a) Fifteen (b) Eight (c) Twelve

28. The best shape of triangle is isosceles with base angles
(a) 61° 30' (b) 56° 14' (c) 52° 25'

29. A dark glass must be employed for the observation of the
(a) Star (b) Sun (c) Neither (a) nor (b)

30. Snellius Rhombus is a quadrilateral with
(a) One diagonal (b)two diagonals (c) Four diagonals

31. The triangles in geodetic survey are
(a) Plane triangle (b) Isosceles triangle (c) Spherical triangle

32. The colliery triangulation is of
(a) First order (b) Second order (c) Third order

33. The triangles in a triangulation network should be
(a) Right angles (b) Ideal (c) Plane

34. For determining the latitude of a survey station the device fitted to an ordinary transit theodolite,which enables to solve astronomical triangle mechanically,is
(a) Chronometer (b) Solar attachment (c) Tellurometer

35. A short-range EDM is one which is capable of measuring distances upto about
(a) 100m (b) 1km (c) 3km.

Answer: 1(b), 2 (c), 3 (c), 4 (c), 5 (b), 6 (a), 7 (b), 8 (b), 9(c), 10 (b), 11(a), 12 (c), 13 (b), 14 (a), 15 (c), 16(a), 17 (c), 18 (b), 19 (c), 20 (c), 21 (a), 22 (b), 23 (c), 24 (a), 25 (a), 26(b), 27 (c), 28 (b), 29 (b), 30 (a), 31 (c), 32 (c), 33 (b), 34 (b), 35 (c).

CORRELATION

SHORT QUESTIONS

Question 1. What is correlation survey ?

Answer : Correlation, in surveying is the method of surveying by which the surface survey and underground survey are connected to the same base.

Question 2. What are the different methods of correlation of surface and under ground surveys ?

Answer : The different methods are : (i) Direct traversing, (ii) Shaft plumbing methods, (iii) Optical method, (iv) Magnetic method and (v) Gyro-theodolite method.

Question 3. What is coplaning method of correlation ?

Answer : The coplaning method, also known as direct alignment method, consists of setting up theodolite in the plane of the two plumb lines in one shaft at surface and underground to transfer the bearing of surface survey directly belowground .

Question 4. Plumb bobs are normally made of lead in shaft plumbing ?

Answer. Plumb - bobs are normally made of lead which has a high density and is unaffected by magnetic attraction.

Question 5. Why are the plumb - bobs immersed in buckets of water or oil in shaft plumbing ?

Answer. To cut down the oscillation of the wire due to (a) irregular mass in strata, (b) simple pendulam motion, (c) air current, and (d) dropping water.

Question 6. What is lateral shift of instrument in coplaning method ?

Answer : The lateral shift is the distance by which the theodolite is to be shifted laterally for exact alignment with plumb-plane from the instrument station.

Question 7. What is Weisbach Triangle ?

Answer. In Weisbach's method of correlation the theodolite is set up slightly out of line forming a small triangle with the two wires. This triangle is known as Weisbach Triangle.

Question 8. In a Weisbach triangle, the Weisbach angle must be very small. Why ?

Answer. If the Weisbach angle is less than 25 minutes, there is no necessity of measuring sides precisely but the angle should be measured very accurately to avoid any error.

Question 9. What are the effects of correlation in two deep shafts far apart.

Answer. In deep shafts far apart, the convergence of the verticals make an appreciable difference to the wire interval belowground as compared with their positions as surface.

Question 10. How will you minimise the twisting motion of the plumb- bob in shaft syrveying.

Answer. The plumb bobs for shaft surveying should be symmetrically shaped and fitted with vanes to dampen the twisting motion when immersed in water.

CHOOSE THE CORRECT ANSWER

1. Where entry to a mine is gained by means of a drift, correlation is carrried out by
(a) Co-planing method (b) Alignment method (c) Direct traversing method

2. Co-planing method of correlation is also known as
(a) Approximate alignment method (b) Exact alignment method
(c) Direct traversing method

3. The permissible error for correlation by wires in two shafts is
(a) ± 1' of arc (b) ± 2' of arc (c) ± 5' of arc

4. Method of correlation using two or more wire in a single shaft is
(a) Weiss quadrilateral (b) Optical transfer (c) Magnetic method

5. The permissible limit of error for direct traverse in correlation is
(a) ± 1 minute of arc (b) ± 2 minute of arc (c) ± 3 minute of arc.

6. In Weiss method, number of theodolite used is
(a) One (b) Two (c) Three

7. In precise magnetic correlation by Holden's method the number of instruments used are
(a) One theodolit (b) Two theodolites (c) Four theodolites

8. The best time for making the magnetic observation is between
(a) 8 a.m. to 10 a.m. (b) 11 a.m. to 2 p.m. (c) 2 p.m. to 4 p.m.

9. The object of co-planing method is to place the axis of theodolite exactly in line with the
(a) Magnetic meridian (b) Line of collimation (c) Plumb plane

10. Plumb bobs used in shaft plumbing is made of
(a) Lead (b) Iron (c) Neither (a) nor (b)

11. Plumb bobs for shaft plumbing are fitted with vanes
(a) To immerse in water (b) To aid verticality of the wire (c) To dompen twisting motion

12. Weisbach method of cerrelation is also known as
(a) Direct alignment method (b) Approximate alignment method (c) Neither (a) nor (b)

13. To make accurate bisection of wires, the diameter of wires should be
(a) Larger (b) Medium (c) Smaller

14. The Weisbach angle should be less than
(a) $1^{\circ}45'$ (b) $1^{\circ}30'$ (c) 25'

15. The surface survey is connected to the underground survey in correlation through
(a) An adit (b) A shaft (c) Both (a) & (b) (d) Neither (a) nor (b)

16. In shaft plumbing, the plumb bobs are immersed in buckets of water
(a) To cut done oscillation of wires (b) To avoid kinking
(c) To avoid effect of ventilation current

17. Correlation by two shaft method, the whole of the underground traverse has to be adjusted by the method of
(a) Pre-pointing (b) Lateral shifting (c) Swinging

18. With Gyrotheodolites direction an underground line situated anywhere in a mine can be determined with an accuracy of about
(a) 30 seconds of arc (b) 25 seconds of arc (c) 12 seconds of arc

19. To mark the river, lake, tank, pond etc. on mine plans so as to take precautions against inundation the survey to be done by
(a) Traversing (b) Levelling (c) Correlation

20. Direct transference of azimuth down the shaft is done by
(a) Direct Traverse method (b) Optical method (c) Coplaning method

21. The most suitable type of wire used in plumbing the shaft is
(a) Aluminium wire 5mm dia. (b) Copper wire 0.3cm dia.
(c) Hard drawn piano wire 0.1cm dia.

Answer : 1 (c) , 2 (b) , 3 (b) , 4 (a) , 5 (a) , 6 (b) , 7 (b) , 8 (b) , 9 (c) , 10 (a) , 11 (c) , 12 (b) 13 (c) , 14 (c) , 15 (c) , 16 (a) , 17 (c) , 18 (c) , 19 (c) , 20 (b) , 21 (c).

PHOTOGRAMMETRY

SHORT QUESTIONS

Question 1. What is Photogrammetry ?

Answer : Photogrammetry is the science of obtainging information about physical objects through process of recording, measuring and interpreting of photographs of the area.

Question 2. What is aerial photogrammetry ?

Answer : Aerial photogrammetry is the branch of photogrammetry in which photograhs of the area are taken by a camera mounted in an aircraft.

Question 3. What is the difference between 'metric photogrammetry' and 'interpretative photogrammetry'

Answer : Metric photogrammetry includes making precise measurements from photographs of the objects to determine the relative location of various points. Distances, angles, areas, volumes, etc. are found in metric system, whereas the interpretative photogrammetry deals with the photo interpretation to recognize and identify objects their significance through careful and systematic analysis.

Question 4. What is the difference between 'space photogrammetry' and 'close-range photorammetry' ?

Answer : Space photogrammetry is the branch of photogrammetry in which photographs are taken by cameras fixed in space. In close-range photogrammetry the camera is kept quite close to the object to obtain the detailed information about the object.

Question 5. What are the types of aerial photographs ?

Answer : Aerial photographs are usually two types (a) Vertical photographs , and (b) oblique photographs.

Question 6. What are basic components of an aerial camera ?

Answer : An aerial camera has three basic components (a) Magazine, (b) camera body, and (c) Lens cone assembly.

Question 7. What are comparators ?

Answer : Comparators are the instruments which are used for a very high accuracy in the measurement of co-ordinates on one photograph or for measurement on an overlapping steriopair of photographs

Question 8. What are the corrections required to the measured photocoordinates ?

Answer : The corrections generally required are (i) Principal point correction, (ii) Shrinkage correction, (iii) Correction for radial lens distortion, (iv) Correction for atmospheric refraction, (v) Correction for earth curvature.

Question 9. What is the scale of the photograph ?

Answer : The scale of the photograph is the ratio of the photo distance and ground distance.

Question 10. What is the scale of the vertical photograph ?

Answer : The scale of a vertical photograph is directly proportional to the focal lenght of the camera and inversely proportional to the flying height above ground.

Question 11. What is relief displacement ?

Answer : Relief displacement is the lateral displacement or shift in the photographic position of the image of an object due to its relief.

Question 12. What is drift of a photograph ?

Answer : Drift is the lateral shifting of the photograph.

Question 13. What is Terrestrial photogrammetry ?

Answer : Terrestrial photogrammetry is a branch of photogrammetry in which photographs are taken with camera lacated on or near the surface of the earth.

Question 14. Name the branches of Terrestrial photogrammetry. ?

Answer : Terrestrial Photogrammetry can be divided into two branches : (i) Plane-table photogrammetry, (ii) Terrestrial stereo-photogrammetry.

CHOOSE THE CORRECT ANSWER

1. Photogrammetry may be devided into
   (a) Two classes (b) Three classes (c) Five classes
2. The system of surveying in which plans or maps are prepared from photographs that are taken from a ground station is known as
   (a)Aerial photogrammetry (b)Terrestrial photogrammetry (c) Aerial surveying
3. Terrestrial photogrammetry is not suitable for
   (a) Flat country (b)Wooded country (c) Both (a) and (b)

4. In aerial camera,lens used is
   (a) Double concave lens (b) Thin convex lens (c) Thick combination
5. The arial camera magazine holds the
   (a) Electric motor for drive mechanism (b) Film (c) Lens assembly
6. Modern aerial cameras are usually equipped with
   (a) Data recorder (b) Spider (c) Cocking shutter
7. A photograph is perspective
   (a) Image (b) Projection (c) Area
8. Scale of photograph is equal to
   (a) $\frac{\text{Photo distance}}{\text{Ground distance}}$ (b) $\frac{\text{Ground distance}}{\text{Photo distance}}$ (c) $\frac{\text{Focal Length}}{\text{Flying height}}$
9. Scale of vertical photograph is equal to
   (a) $\frac{\text{Ground distance}}{\text{Focal length}}$ (b) $\frac{\text{Focal Length}}{\text{Flying above ground dist.}}$ (c) $\frac{\text{Focal Length}}{\text{Photo dist.}}$
10. The relief is the elevation of the object above or below a
    (a) Camera (b) Magazine (c) Datum
11. Flying height may be measured approximately with
    (a) Altimeter (b) Clinometer (c) Kilometre
12. The angle between flight line and the adge of the photographs in the direction of flight is
    (a) drift (b) Crab (c) Relief
13. A steriometric camera consists of
    (a) One camera (b) Two cameras (c) Three cameras
14. The most common application of photogrammetry is in the topgraphic mapping of
    (a) Small areas (b) Madium areas (c) Large areas
15. In a titled photograph the tilt usually less than
    (a) 1^0 (b) 2^0 (c) 3^0
16. The side overlap of steriopair of photographs is usually less than
    (a) 30% (b) 40% (c) 50%

Answer : 1 (a) 2 (b), 3 (c), 4 (c), 5 (b), 6 (a), 7 (b), 8 (a), 9 (b), 10 (c), 11(a), 12 (b), 13 (b), 14 (c), 15 (c) , 16 (a) ,

CHAPTER VIII

NUMERICAL EXAMPLES

Example 1. The following are notes of an underground theodolite traverse :

Line	Azimuth Bearing	Distance (m)	Vertical Angle
AB	89° 54'	350	Level
BC	150° 12'	190	Level
CD	180° 00'	600	Level
DE	140° 18'	155	+ 28° 00'
EF	228° 36'	800	- 12° 00'

It is proposed to drive a cross-measures drift to connect A with F. Calculate the length, gradient and bearing of the lines FA.

Answer : Before calculations can commence, the azimuth bearings of the survey lines have to be converted to quadrant bearings. The calculations of Latitude and Departure can be tabulated ---

Line Stn.	Distance mts.	Azimuth	Quadrant Bearing 0	Partial Latitude L cos 0	Partial Departure L sin 0	Total Co-ordinates	
						Lat.	Dep.
A				0	0	0	0
AB	350	89° 54'	N 89° 54' E	+ 0.61	+ 350.00	+ 0.61	+ 350.00
B							
BC	190	150° 12'	S 29° 48' E	- 164.90	+ 94.43	- 164.29	+ 444.43
C							
CD	600	180°.00'	Due South	- 600.00	---	- 764.29	+ 444.43
D							
DE	155	140° 18'	S 39° 42' E	- 105.30	+ 87.40	- 869.59	+ 531.83
E							
EF	800	228° 36'	S 48° 36' W	- 517.50	---	- 1387.09	- 55.07
F							

Therefore, total co-ordinates of Station F are :

Latitude : South 1387.09 m.

Departure : West 55.07 m

$$\text{Tan bearing FA} = \frac{\text{Departure}}{\text{Latitude}} = \frac{55.07}{1387.09}$$

Bearing FA = 2° 16'

Plan length FA = Departure of F x secant bearing

55.07 secant 2° 16' = 1392.

Therefore, plan length of FA (proposed drift) = 1392m .

In order to calculate the actual (inchined) length of the drift, it is necessary to find the difference in level between A and F , as under :

Rise on line DE = 155 x Sin 28° 00' = 72.76 m.

Fall on line EF = 800 x Sin 12° 00' = 166.30 m.

Difference in level between A and F = 166.30 - 72.76 = Fall of 93.54 m.

$$\text{Tangent of angle of inclination of drift} = \frac{93.54}{1392.00}$$

Inclination of drift = 3° 51'

This is equivalent to a gradient of 1 in 14.86

Therefore, drift will rise at 1 in 14.86 from F to A.

$$\text{True length of drift} = \frac{1392.00}{\text{Cosine } 3° 51'} = \mathbf{1395\ m}$$

Example 2. The following data refer to a theodolite survey which was made to enable a straight road to be driven from a point A to another point D. Calculate the rectangular co-ordinates of the stations B,C and D relative to the point A as origin, and find the bearing and length of the line AD.

Line	*Azimuth*	*Horizontal angle*	*Distance*
AB ...	47°42'	---	384.9m
BC...	---	ABC : 176° 54'	473.5m
CD...	---	BCD : 185° 32'	321.6m

Answer : To find Azimuth Bearing of BC and CD ---

Azimuth BC = 47° 42' + 176° 54' - 180° 00' = 44° 36'

Azimuth CD = 44° 36' + 185° 32' - 180° 00' = 50° 08'

To Find Quadrant Bearing of AB, BC and CD :

AB Azimuth 47° 42' = N 47° 42' E.

BC Azimuth 44° 36' = N 44° 36' E.

CD Azimuth 50° 08' = N 50° 08' E.

To Find Rectangular Co-ordinates :

Latitude AB = 384.9 x cos 47° 42' = 384.9 x 0.6730 = 259.0377 N.

Latitude BC = 473.5 x cos 44° 36' = 473.5 x 0.7120 = 337.1320 N.

Latitude CD = 321.6 x cos 50° 08' = 321.6 x 0.6410 = 206.1456 N.

Departure AB = 384.9 x sin 47° 42' = 384.9 x 0.7396 = 284.6720 E.

Departure BC = 473.5 x sin 44° 36' = 473.5 x 0.7022 = 332.4917 E.

Departure CD = 321.6 x sin 50° 08' = 321.6 x 0.7676 = 246.8601 E.

The result are tabulated as follows, and the total latitudes and departures gives the co-ordinates of points B, C and D in relation to original point A .

Line	Dist. m.	Quad. Bearing	Latitudes N.	Latitudes S.	Departures E.	Departures W.	Total Lats.	Total Deps.
AB	384.9	N47°42'E	259.0377	---	284.6720	---	259.0377N	284.6720E
BC	473.5	N44°36'E	337.1320	---	332.4917	---	596.1697N	617.1637E
CD	321.6	N50°08'E	206.1456	---	246.8601	---	802.3153N	864.0238E
			802.3153		864.0238			

$$\text{Tan bearing} = \frac{\text{departure}}{\text{latitude}} = \frac{864.0238}{802.3153} = 1.0760$$

Tan 10.769 = 47°07' and bearing of AD = **N 47°07'E.**

Length AD = $\sqrt{864.0238^2 + 802.3153^2}$ = **1179.08 m.**

Exapmle 3. The co-ordinates of two stations B and C in relation to another station A and as follows :

Station	*Latitude*	*Departure*
B	N 576	E 215
C	S 750	E 800

Calculate the radius of the circular curve passing through stations A, B and C.

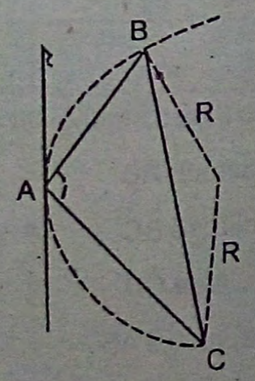
Answer : Latitude of A = 0.00 Departure of A = 0.00.

$$\text{Bearing of AB} = \tan^{-1}\left(\frac{\text{Dep. of B - Dep. of A}}{\text{Lat. of B - Lat. of A}}\right)$$

$$= \tan^{-1}\left(\frac{215 - 0}{576 - 0}\right) = \text{N } 20°28'08'' \text{ E}$$

$$\text{Bearing of AC} = \tan^{-1}\left(\frac{\text{Dep. of C - Dep. of A}}{\text{Lat. of C - Lat. of A}}\right)$$

$$= \tan^{-1}\left(\frac{800 - 0}{-750 - 0}\right) = \text{S } 46°50'51'' \text{ E.}$$



Length of BC
$= (800 - 215)^2 + (-750 - 576)^2 = 1449.311\text{m}$

$\angle A = 180° - (20°28'08'' + 46°50'51'') = 112°41'01''$

$$\text{By Sine Rule, } R = \frac{a}{2 \sin A} = \frac{BC}{2 \times \sin 112°41'01''}$$

$$= \frac{1449.311}{1.8452969}$$

$= 785.408$ m.

Radius of the curve = 785.408m

Example 4. Plot the following survey by co-ordinates and calculate the length and bearing of the closing line :

Line	Magnetic Bearing	Distance in m.
AB	30°	900
BC	42°	610
CD	233°	750

Answer : Assuming the co-ordinates of station A to be latitude = 0 metre and departure = 0 metre, the co ordinates of other stations are tabulated below ---

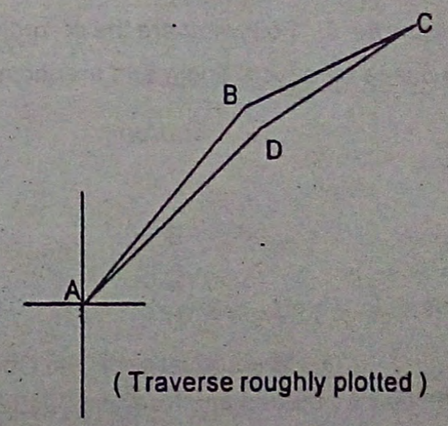
Stn. Line	Alzimuth °	Length L m	Q.B.	Partial Latitude (L.cos θ)	Partial Departure (L.sin θ)	Total Latitude	Total Departure
A						0.000	0.000
AB	30°	900	N 30° E	+779.423	+450.000		
B						+779.423	+450.000
BC	42°	610	N 42° E	+453.318	+408.170		
C						+1232.423	+858.170
CD	233°	750	S 53° W	-451.361	-598.977		
D						+781.380	+259.193

Length of AD $= \sqrt{259.193^2 + 781.380^2}$

= 823.247m.

Bearing of AD $= \tan^{-1}\left(\frac{259.193}{871.380}\right)$ NE

= N 18° 21' 4.66" E



(Traverse roughly plotted)

Example 5. Given the following co-ordinates of the stations at the corners of a five sided enclosure, find the area of the field in square metres.

Stations	*Latitude*	*Departure*
A	N 650	E 375
B	S 475	E 520
C	S 240	W 600
D	N 330	W 100
E	N 600	W 450

Answer : By applying the Double Area Method, the rule of which states : The area is equal to one half the algebraic sum of the products obained by multiplying the total departure of each station by the difference in total latitude of adjacent stations.

Thus, starting at station A, then to B, to c, etc. we have :

Double Areas

375 x [600 - (- 475)] = 375 x 1075 = + 403,125

520 x [650 - (- 240)] = 520 x 890 = + 462,800

- 600 x [- 475 - 330] = - 600 x -805 = + 483,000

- 100 x [- 240 - 600] = - 100 x -840 = + 84,000

- 475 x [330 - 650] = - 475 x -320 = + 144,00

Doube Area = sum of products = 1,576,925 sq.m.

Area = 1,576,925 ÷ 2 = **788462.5 sq.m.**

Example 6. Following are the co-ordinates in feet of certain stations of a colliery survey which is based on a local origin and meridian :

Station	*Easting (feet)*	*Northing (feet)*
A	6585.33	8966.52
B	9931.43	4792.70
C	7083.43	2568.63
D	2130.84	1993.49

The full National Grid Co-ordinates of Station A have been determined as Easting 299060.53 metres and Northing 204961.85 metres and of Station D as Easting 297661.24 and Northing 202594. 43 metres. The National Grid Bearing of the line AD has been determined as 213°42' 33". The above co-ordinates do not refer to actual stations.

The position of the National Grid is such that the distances from which the colliery survey co-ordinates have been computed require to be reduced by 1/ 3620 to conform to National Grid distances.

In converting feet to metres multiply by 0.3048007 of which the logarithm is 1.4840160.

From the above data calculate the full National Grid Co-ordinates, to two places of decimals, of the station B and C

Answer : To find the azimuth of the line AD from the co-ordinates of the colliery survey ---

Departure of D reletive to A = 6585.33 - 2130.84
= 4454.49W.

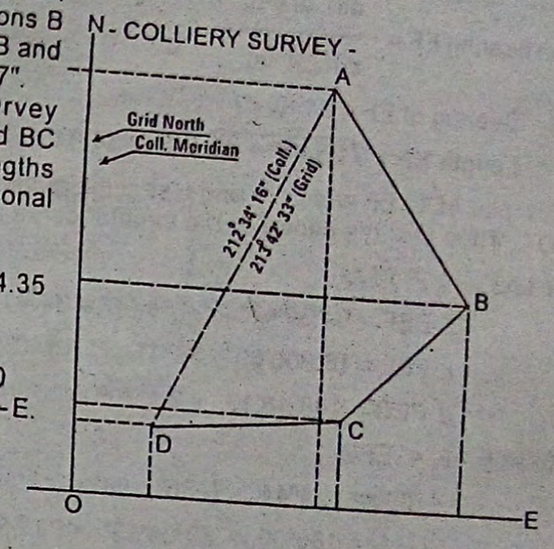
Latitude of D relative to A = 8966.52 - 1993.49
= 6973.03S.

Quadrant bearing of line AD = S $\tan^{-1}\frac{4454.49}{6973.03}$ W.

Azimuth of line AD = S 32°34' 16" W.
= 180° + 32° 34' 16"
= 212°34' 16".

The angle between Grid North and meridian of the colliery survey = 213°42'33" - 212°34'16" = 1°08'17".
As the National Grid Co-ordinates of the stations B and C are to be found the azimuths of lines AB and BC must be corrected by the addition of 1°08'17".

As the co-ordinates of the colliery survey stations are in feet the lengths of lines AB and BC must be reduced to metres, after which the lengths must diminished by 1/3620 to conform to National Grid distances.



Length of AB = $\frac{3619}{3620}\sqrt{3346.10^2 + 4173.8^2}$ = 5584.35 feet, or 1702.11 metres.

Quardrant bearing of line AB = S $\tan^{-1}\frac{3346.10}{4173.82}$ E.
= S 38°43'07" E.

Corresponding azimuth of line AB = 141°16'53"

Grid bearing of line AB = 141°16'53" + 1008'17"
= 142°25'10".

Quadrant bearing reduced to Grid North = S37°34'50" E.

Easting of B relative to A = 1702.11 sin 37°34'50"
= 1037.82 metres.

Southing of B relatives to A = 1702.11 cos 37°34'50"
= 1660.20 metres.

Grid co-ordinates of station B : Easting, 300098.35 metres ; Northing, 203031.50 metres.

Length of BC = $\frac{3619}{3620}\sqrt{2848.00^2 + 2223.87^2}$ = 3612.55 feet, or 1101.62 metres.

Quadrant bearing of line BC = S tan-1 $\frac{2848.00}{2223.87}$ W.
= S52°00'58" W.

Corresponding Azimuth of line BC = 232°00'58".

Grid bearing of line BC = 232°00'58" + 1°08'17" = 233°09'15".

Quadrant bearing to Grid North = S53°09'15" W.

Westing of C relative to B = 1101.62 sin 53°09'15"
= 881.60 metres.

Souting of C relative to B = 1101.62 cos 53°09'15"
= 560.60 metres.

Grid co-ordinates of station C : Easting, 299216.75 metres; Northing , 202370.90 metres.

Example 7. Two haulage roadways EB and CFD in a mine, whose azimuths are respectively 24°58' and 324°44' , are connected by a straaight roadway driven from E to F. The co-ordinates of the point E and F are as follows ---

	Latitude	Departure
E	226.25m	193.11m
F	249.71m	242.65m

It is proposed to drive a curved road which shall be tangential to each of lines AE, EF and CF. Calculate the radius of the curve.

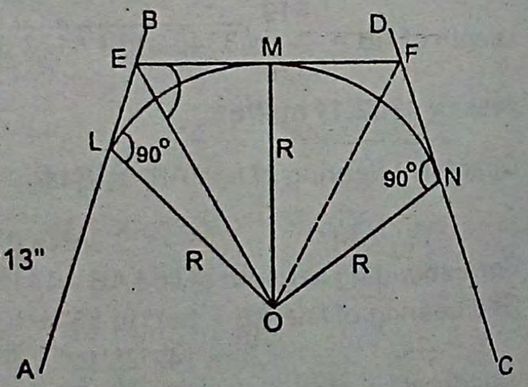
Answer : To calculate the length and bearing of EF ---

$$\text{Tan bearing EF} = \frac{\text{diff. of dep.}}{\text{diff. of lat.}} = \frac{242.65 - 193.11}{249.71 - 226.25} = \frac{49.54}{23.46} = 2.112$$

Bearing of EF = 64°39'47"

Length EF = $\sqrt{23.46^2 + 49.54^2}$ = 54.814m.

Now plot AEB, EF and CFD and bisect the angles at E and F, the bisectors meeting at centre ' O ' . Then O is the centre of the circular curve and the radius = OL = OM = ON = 45.67m.



To find α = FEO .

$\angle$ BEF = 64°39'47" - 24°58'00" = 39°41'47"

$\angle$ FEL = 180°00'00" - 39°41'47" = 140°18'13"

$\angle$ FEO = 140°18'13" ÷ 2 = 70°09'06" = α

To find θ = EFO

$\angle$ EFD = 324°44' - (180° ÷ 64°39'47") = 80°04'13"

$\angle$ EFN = 180°00' - 80°04'13" = 99°55'47"

$\angle$ EFO = 99°55'47" ÷ 2 = 49°57'53" = θ

Also EOF = 180°00'00" - (α + θ) = 180° - 120°06'59" = 59°53'01" = EOF

$$\text{Now, in triangle EOF. OF} = \frac{\text{EF Sin } \alpha}{\text{Sin EOF}} = \frac{54.814 \text{ sin } 70°09'06"}{\text{Sin } 59°53'01} = 59.604\text{m}$$

In triangle MOF, **Radius** = R = OM = OF Sin θ

= 59.604 Sin 49°57'53" = **45.636m**

Example 8. Calculate the latitudes , departures and closing error for the following traverse, and adjust using the Bowditch's Rule.

Line	*Length (m)*	*Whole Circle Bearing*
AB	89.31	45°10'
BC	219.76	72°05'
CD	151.18	161°52
DE	159.10	228°43'
EA	232.26	300°42'

Answer : Calculations are given below ---

Line	W.C.B	length (m)	Latitude			Departure		
			Calculated	Correction	Corrected	Calculated	Correction	Corrected
AB	45°10'	89.31	+ 62.97	- 0.06	+ 62.91	+ 63.34	- 0.02	+ 62.32
BC	72°05'	219.76	+ 67.61	- 0.13	+ 67.48	+ 209.10	- 0.06	+ 209.04
CD	161°52'	151.18	- 143.67	- 0.09	- 143.78	+ 47.05	- 0.04	+ 47.01
DE	228°43'	159.10	- 104.97	- 0.10	- 105.07	- 119.56	- 0.04	- 199.60
EA	300°42'	232.26	+ 118.58	- 0.14	+ 118.44	- 199.71	- 0.06	- 199.77
	Σ	851.61	+ 0.52	- 0.52	0.00	+ 0.22	- 0.22	0.00

Corrections : For latitude of AB = $\frac{-0.52}{851.61} \times 89.31 = -0.06$ m

For departure of AB = $\frac{-0.22}{851.61} \times 89.31 = -0.02$ m

Corrections for other lines are determined using the same procedure .

Closing Error = $\sqrt{(0.52)^2 + (0.22)^2}$ = 0.565m.

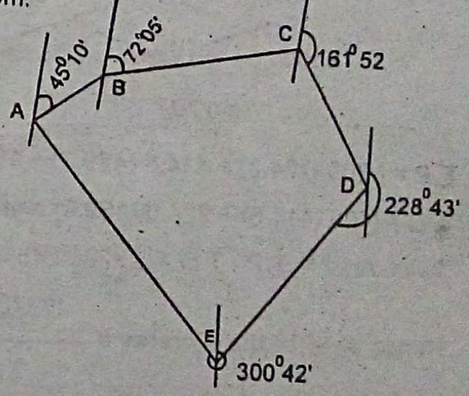
The angle of the error of closure (θ) is given by

$$\text{Tan } \theta = \frac{0.22}{0.52} = 0.4231$$

θ = 22°56'

Relative accuracy

$$= \frac{0.565}{851.61} = 1 \text{ in } 1507$$



Example 9 : The following table gives the latitudes and departures of the sides of a closed traverse ABCD :

Line	Latitude		Departure	
	North	South	East	West
AB	214.8m		124.0m	
BC		245.1m	205.7m	
CD		155.9m		90.0m
DA	186.2m			239.7m

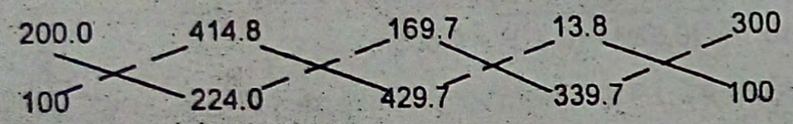
Compute its area.

Answer : The area of the closed traverse ABCD to be computed by the application of the foregoing rule and the results tabulated below ---

Line	Lat. (m)	Dep. (m)	Stn.	Independent Co-ordinates	
				North 'y' (m)	East 'x' (m)
AB	+ 214.8	+ 124.0	A	200.0	100.0
BC	- 245.1	+ 205.7	B	414.8	224.0
CD	- 155.9	- 90.0	C	169.7	429.7
DA	+ 186.2	- 239.7	D	13.8	339.7
			A	200.0	100.0

The north co-ordinates of the stations B,C and D are obtained by first assuming the north co-ordinate of A as 200 and then successively adding algebraically the latitude of each of the other stations. Similarly , the east co-ordinate of A is assumed as 100 and the east co-ordinates of the other stations obtained by successively adding algebraically their departures.

Arranging the co-ordinates in the determinant from as follows ----



$\Sigma P = \{ 200\ (224.0) + 414.8\ (429.7) + 169.7\ (339.7) + 13.8\ (100.0)\} = 282066.45.$

$\Sigma Q = \{100\ (414.8) + 224.0\ (169.7) + 429.7(13.8) + 339.7\ (200.0)\} = 153362.64.$

Twice Area = $\Sigma P - \Sigma Q = 282066.45 - 153362.64 = 129703.81$

Hence, **Area of the traverse** = $\frac{129703.81}{2}$ = **64,351.90 sq.m**

Example 10 : The following latitude and departures in meteres are of a closed traverse :

Line	Latitude	Departure
AB	- 232.2	- 88.8
BC	+ 13.6	+ 116.4
CD	+ 161.0	+ 34.4
DA	+ 57.6	- 62.0

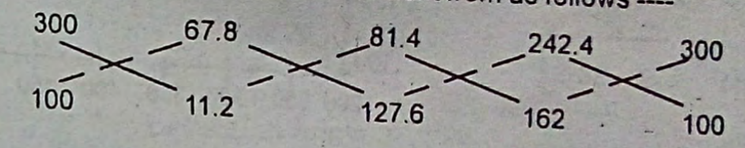
Plot the traverse on a scale 20m. to 1cm. and also calculate the area of traverse.

Answer : TO PLOT TRAVERSE AND CALCULATE THE AREA

The area of the closed traverse ABCD will be computed by the application of the foregoing rules and the results tabulated as under :

Line	Lat.	Dep.	Stn.	Independent Co-ordinates	
				North	East
AB	- 232.2	- 88.8	A	300.0	100.0
BC	+ 13.6	+ 116.4	B	67.8	11.2
CD	+ 161.0	+ 34.4	C	81.4	127.6
DA	+ 57.6	- 62.0	D	242.4	162.0
			A	300.0	100.0

Arranging the co-ordinates in the determinant from as follows ----



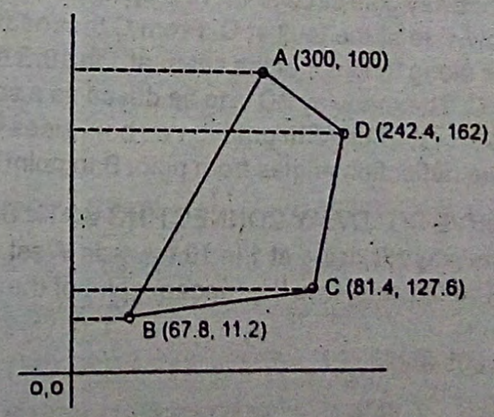
$\Sigma P = \{ 300\ (11.2) + 67.8\ (127.6) + 81.4\ (162) + 242.4\ (100)\} = 49438.08$

$\Sigma Q = \{100\ (67.8) + 11.2\ (81.4) + 127.6\ (242.4) + 162\ (300)\} = 87221.92$

Twice = $\Sigma P - \Sigma Q = 49438.08 - 87221.92 = 37783.90$ sq.m.

Area = 37783.90 ÷ 2 = 18891.95 sq.m.

Plotted the traverse on a scale 20m to 1 cm as follows :



Example11. The bearings of two inaccessible stations A and B taken from C were 225°0' and 153°26" respectively. The co-ordinates of A and B were as under :

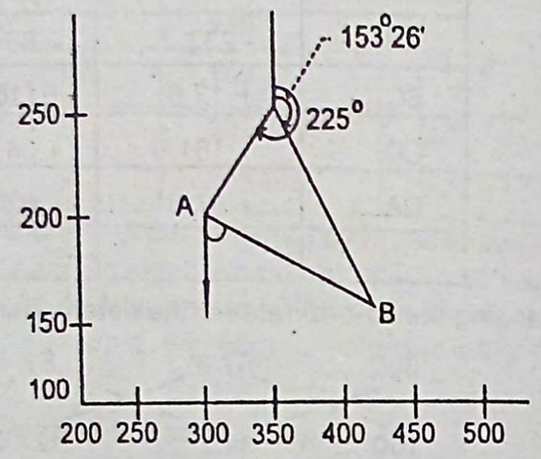
Station	*Easting*	*Northing*
A	300m	200m
B	400m	150m

Calculate the independent Co-ordinates of C.

Answer : Length of AB $= \sqrt{(400 - 300)^2 + (200 - 150)^2} = 111.80$ m.

$$\text{Tan} \propto = \frac{100}{50} = 2.0 \quad \text{Or,} \quad \propto = 63°26' = 116°34'$$

The W.C. Bearing of the line AB is $180° - 63°26' = 116°34'$.
In triangle ACB,



$\angle CAB = 116°34' - 45° = 71°34'$
$\angle ACB = 225° - 153°26' = 71°34'$
$\angle ABC = 180° - (71°34' + 71°34') = 36°52'$

By Sine law,

$$\frac{111.80}{\text{Sin } 71°34'} = \frac{BC}{\text{Sin } 71°34'} = \frac{CA}{\text{Sin } 36°52'}$$

CA = 70.70m

Latitude of CA = 70.70 cos 225° = - 50 m

Departure of CA = 70.70 sin 225° = - 50m

Independent co-ordinates of C, Easting = 300 + 50 = **350 m.E**
Northing = 200 + 50 = **250 m. N**

Example 12 : A roadway AB rises at a gradient of 1 in 10 in a westerly direction along the floor of a seam, the full dip of which is due south at 1 in 5. It meets a fault at B which throws the seam 7.5m down to the west. The line of intersection of fault and seam is due North and South . It is proposed to continue the roadway on a circular curve dipping at a gradient of 1 in 5 to the floor of the seam on the down throw side of the fault at C. From C the roadway will continue straight to a point D such that CD dips along the floor of the seam at 1 in 10. Thus the tangent points of the circular curve are at B and C. The roadway BC is to be driven as a series of chords , length 7.5m plus final sub-chord. The seam is of uniform gradient on both sides of fault. Calculate the radius of the circular curve and the deflection angles from point B to point C.

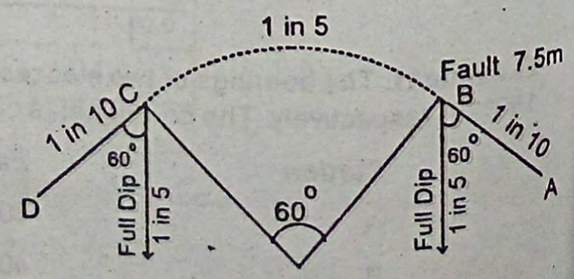
ANSWER : CIRCULAR CURVE ROADWAY CONNECTING BOTH SIDES OF A FAULTED SEAM

The position of roadway AB rising at 1 in 10 towards West and CD dipping along the floor of the seam 1in 10. The figure shows the tangent point of the circular curve at B and C.

Tan apparent dip
= tan full dip x cos angle between dips.

$$\frac{1}{10} = \frac{1}{5} \times \cos \propto \; ; \; \propto = 60°$$

$$\text{Plan length of sub-chord} = \frac{37.5}{\sqrt{26}} = 7.5348\text{m}.$$



$$\text{Vertical displacement of each chord} = \frac{7.5}{\sqrt{26}} = 1.471\text{m}.$$

Vertical difference between B and C = 7.5

$$\text{Number of sub - chords} = \frac{7.5}{1.471} = 5.098$$

Four sub - chords of 7.3548m plan length.

One sub - chord of 8.0826 plan length.

$$\text{Arc} = 2\pi R \times \frac{\theta}{360} \quad \text{or} \quad 37.501 = 2\pi R \times \frac{60}{360} = \frac{\pi R}{3}$$

$$R = \frac{112.502}{\pi} = 35.75 \text{ m.}$$

Radius of curve = 35.75

Now, sine deflection angle (1) $= \frac{\text{Sub-chord length}}{2R} = \frac{7.3548}{35.75 \times 2}$

Deflection Angle = 5°54' ..... (1)

Sine Deflection angle (2) $= \frac{8.0826}{35.75 \times 2}$

Deflection Angle = 6°29' ..... (2)

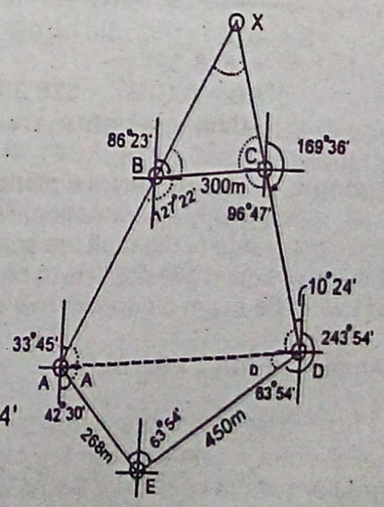
Example 13: A closed traverse was conducted round an obstacle and the following observations were made. Work out the missing quantities.

Side	Length	Azimuth
AB	—	33°45'
BC	300	86°23'
CD	—	169°36'
DE	450	243°54'
EA	268	317°30'

Answer : Interior Angles

$\angle B = (180° - 86°23') + 33°45' = 127°22'$
$\angle C = (180° - 169°36') + 86°23' = 96°47'$
$\angle D = 360° - (243°54' + 10°24') = 105°42''$
$\angle E = (360° - 317°30') + (243°45' - 180°) = 106°24'$
$\angle A = 180° - [(33°45') + (360° - 317°30')] = 103°45'$

Total interior angles = 540°00'



In triangle AED, $AD^2 = AE^2 + ED^2 - 2AE \times ED \cos 106°24'$

$= 268^2 + 450^2 - 2 \times 268 \times 450 \cos 106°24'$

$= 342424.76$

$AD = \sqrt{342424.76} = 585.17\text{m}.$

Now, $\frac{\text{Sin A'}}{450} = \frac{\text{Sin } 106°24'}{585.17}$ or, $\text{Sin A'} = \frac{\text{Sin } 106°24' \times 450}{585.17}$

Angle A' = 47°32'

and $\frac{\text{Sin D'}}{268} = \frac{\text{Sin } 47°32'}{450}$ or, $\text{Sin D'} = \frac{\text{Sin}47°32' \times 268}{450}$

Angle A' = 47°32'

$\angle ADC = 105°42' - 26°04' = 79038'$

$\angle BAD = 103°45' - 47°32' = 56°13'$

The lines AB and DC are produced to intersect at X shown in figure.

$\angle XAD = 103°45' - 47°32' = 56°13'$ and $ADX = 105°42' - 26°24' = 79°38'$

$\angle BCX = 180° - 96°47' = 83°13'$ and $XBC = 86°23' - 33°45' = 52°38'$

$\angle AXD = BXC = 180° (83°13' + 52°38') = 44°09'$

By sine Rule

$\frac{BX}{\text{Sin BCX}} = \frac{BC}{\text{Sin BXC}}$ $\therefore BX \frac{300 \times \text{sin } 83°13'}{\text{Sin } 44° 09'} = 427.6858\text{m}$

$\frac{CX}{\text{Sin XBC}} = \frac{BC}{\text{Sin BXC}}$ $\therefore CX = 300 \times \text{Sin } 52°38' = 342.30717\text{ m}$

$\frac{DX}{\text{Sin } 56°13'} = \frac{AD}{\text{Sin } 44°09'}$ $\therefore DX = \frac{AD \times \text{Sin}56°13'}{\text{Sin } 44°09'}$

$= \frac{585.17 \times \text{Sin } 56°13'}{\text{Sin } 44°09'} = 698.2548\text{ m.}$

$DC = DX - CX = 698.2548 - 342.30715 = 355.947$ metres

$\frac{AX}{\text{Sin ADX}} = \frac{AD}{\text{Sin } 44°09'}$ $\therefore AX = \frac{AD \times \text{Sin ADX}}{\text{Sin } 44°09'} = \frac{585.17 \times \text{Sin } 79°38'}{\text{Sin } 44°09'}$

$= 826.397$ m

$AB = AX - BX = 826.397 - 427.686 = 398.711$ metres

Missing lengths are : AB = 398.711m and DC = 355.947m

Example 14. A coal seam dipping at 1 in 4 in the direction S 300 W encounters a down throw fault, running in E - W direction. The vertical displacement is 50m with its hade of 30°. On the downthrow side of the fault the seam dips 1 in 5 in the direction S45° W. A stonedrift dipping 1 in 2 in the direction due south is to be driven from the upthrow side of fault, 24m from the fault plane to touch the seam on downthrow side. Calculate the length of the stone drift.

Answer : FAULT PROBLEM

The drift is driven due South, i.e. at right angle to the strike of the fault. But as the drift is driven due South and the full dip of the seam on the upthrow side S30° W, 1in 4, hence the gradient of the seam due South will be-

$\frac{1}{4} \cos 30° = \frac{1}{4.62}$ or 1 in 4.62

Again, the full dip on the downthrow side is changed to S45° W, 1 in 5, so the seam gradient due South will be :

$\frac{1}{5} \cos 45° = \frac{1}{7.071}$ or 1 in 0.071

Want of the fault = 50 x tan 30° = 28.867 metres.

AA' = Difference of R.L. A and B = $\frac{24}{4.62} = 5.195$m

BB' = Vertical displacement = 50m.

Let ED = X metres. Now ---

EE' = Diff. of R.L. E and D = E'D + 7.071 = $\frac{X}{7.071}$

$\therefore AD' = 5.195 + 50 + \frac{X}{7.071}$ and DD' = 24 + 28.867 + X

As the drift gradient is 1 in 2 :

$2 \times AD' = DD'$

$\therefore 2(5.195 + 50 + \frac{X}{7.071}) = 24 + 28.867 + X$

or $10.390 + 100 + \frac{2}{7.071} X = 24 + 28.867 + X$

or $X - \frac{2}{7.071} X = 10.390 + 100 - 24 - 28.867$

$0.717 X = 57.523$ or X = 80.227 metres.

Horizontal length of the Drift = 24 + 28.867 + 80.227 = 133.094m

Incilined **length of drift** dipping 1 in 2,

AD = 133.10 x $\frac{\sqrt{5}}{2}$ = 148.81 metres.

Example 15. In making a survey of colliery yard, it is found that the base line intersects the inner or concave side of a curved rail at two points, A and B, 104m. apart, and the offset CD midway between these two points to the side of the rail is 17.33m. Calculate the radius of the curve and the length of the rail between the two points intersected by the base line.

Answer : In the adjoining sketch, let r be the rise CD = 17.33m, and let L= the half - chord AC = 52m.

Let R= the radius = OA = OD = OB

In triangle AOC = $R^2 = OC^2 + AC^2 = (R-r)^2 + L^2$

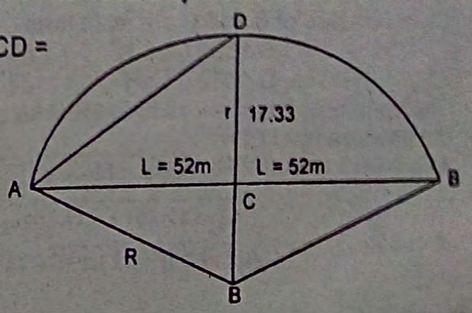
$= R^2 - 2Rr + r^2 + L^2$

Hence $2Rr = r^2 + L^2$

Substituting known values,

$2R \times 17.33 = 17.33^2 + 52^2$

$= 17.33^2 (1 + 3^2) = 3003.3$m



$$\text{Radius} = R = \frac{3003.3}{2x1733^2} = 86.65\text{m.}$$

To find length of arc ADB :

$\text{Sin}\theta = AC \div OA = 52 \div 86.65 = 0.6$. Hence $= 36^0 52'$ and $\angle AOB = 73^0 44'$

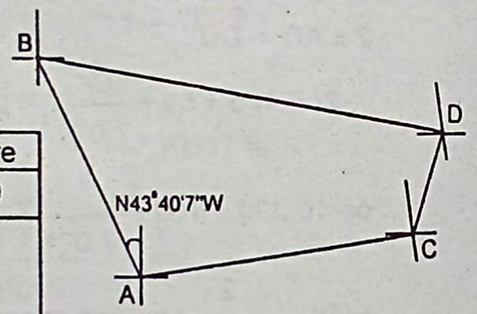
$$\text{Length of rail} = 2\pi R x \frac{73.44'}{360} = 2x3.1416x86.65x \frac{73^0 44'}{360}$$

$$= 111.51\text{m.}$$

Example 16. The length and bearing of line AB can not be observed direct in field due to some obstacle in traversing. The following data are obtained by observation on stations C and D of a traverse ABCD. Calculate the length and bearing of AB.

Line	Fore Bearing	Length
CA	258°30'	235m
CD	10°02'	114m
DB	276°30'	454m

Answer : Fore bearing of line CA = 258°30'
Bearing of AC=258°30'-180°.
= 78°30'



Stn. Line	Azimuth	Distance m	Latitude LxCosθ	Departure Lx Sinθ
A. AC C	78°30'	235	0	0
CD D	10°02'	114	+112.257	+19.861
DB B	276°30'	454	+51.394	- 451.082
			+210.502	- 200.939

Length of AB $= \sqrt{(+210.502)^2 + (-200.939)^2} = 291.011\text{m}$

Bearing of AB $= \text{N.tan}^{-1} \frac{(200.939)}{210.502}$ - W = N 43°40' 07" W.

Example 17. Two underground roadways AB and CD are to be connected by a curved roadway having a radius of 61m. The azimuth of AB and CD are respetively 21°50' and 300° 10'. Co- ordinates of B and D are as follows :

B 1539.05m N 537.26m W
D 1552.21mN 385.26m W.

Calculate now far each roadway must be driven to meet the tangent points and the length of the new roadway BD.

Answer : Length BD= $\sqrt{(\text{Dep. of D -. of B})^2 + (\text{Lat.of D -Lat.of B})^2}$

$\sqrt{(-385.26) + 537.26)^2 + (1552.21+1539.05)^2}$

$\sqrt{(152.00)^2 + (13.16)^2} = 152.569\text{m}$

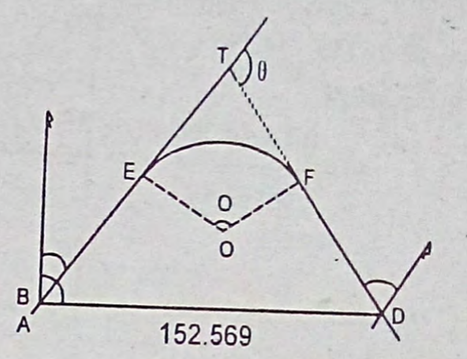
$$\text{Bearing of BD} = \tan^{-1}\left(\frac{\text{Dep. of D- Dep. of B}}{\text{Lat. of D- Lat.of B}}\right)$$

$$= \tan^{-1}\left(\frac{+152.00}{+13.16}\right)$$

$= N\ 85^0 03''\ E$

Deflection angle, θ
$= 180^0 - (21^0 50 + 59^0 50') = 98^0 20'$

$\angle \frac{\theta}{2} = 49^0 10'$



ET=FT= tangent distance

$\angle BTD = 180^0 - 98^0 20' = 81^0 40'$

Tangent Distance $= R \tan \frac{\theta}{2} = 61 \tan 49^0 10' = 70.586\text{m.}$

$\angle DBT = 85^0 03' 6'' - 21^0 50' = 63^0 13' 06'$

$\angle BDT = 180^0 - (85^0 03' 6'' + 59^0 50') = 35^0 06' 54''$

In trangle BTD apply sine Rule,

$$BT = \frac{BD x \sin BDT}{\text{Sin BTD}} = \frac{152.559 x \sin 35^0 06' 54''}{\text{Sin } 81^0 40'} = 88.697\text{m.}$$

$$DT = \frac{BD \times \text{Sin DBT}}{\text{Sin BTD}} = \frac{152.559 \times \text{Sin } 63^0 13' 06''}{\text{Sin } 81^0 40'} = 137.656\text{m.}$$

BE = 88.697-70.586=18.111m
DF= 137.656-70.586= 67.07m
Length of the new road = 152.569m.

The roadway AB is driven 18.111m from B to reach tangent point at E, and the roadway CD is driven 67.07m from D to reach tangent point at F.

Example 18. Two underground roadways AB and CD are to be connected by a curved roadway having a radius of 66.67m. The azimuths of AB and CD are respectively 21°50' and 300°10, and the co-ordinates of B and D are as follows :

B	N1683.0m.	W 387.56m.
D	N1700.8m.	W 421.32m

Calculate (a)how for each roadway must be driven to meet the tangent points, and (b) the length of the new roadway BD.

Answer : To calculate the length and bearing of BD -
Difference of latitude = 17.68m.N.Difference of Departure = 166.3m.E.

Length BD $= \sqrt{17.68^2 + 166.23^2} = 167.17\text{m.}$
Tan bearing = 166.23 ÷ 17.68 = 9.4021 ∴ Bearing BD =83°56' NE.
Now plot AB, BD and CD to scale and produce the straights to interest at X, Then solve triangle BDX to find BX and XD.
Apex angle ϕ = BXD = azimuth XB - azimuth XD
$= 201^0 50' - 120^0 10' = 81^0 40'$

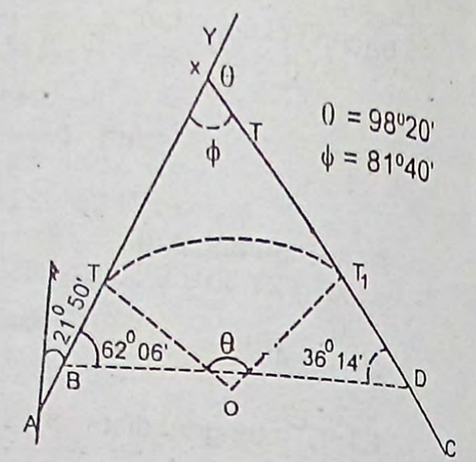
Angle B = azimuth BD-azimuth BX
= 83°56' - 21°50'=62°06'
Angle D =180° - 143°46'=36°14'

In triagle BXD

$$BX = \frac{BD \times \sin 36°14'}{\text{Sin } 81°40'} = \frac{167.17 \times \sin 36°14'}{\sin 81°40'}$$

= 99.86m.

$$XD = \frac{BD \times \sin 62°06'}{\sin 81°40'} = 149.31\text{m}.$$



Intersection angle θ = YXD = TOT, = 180° - 81°40' = 98°20'

Tangent distance TX = XT, = R x tan 49°10' = 66.67 x tan 49°10' = 77.13m

(a) **Distance AB** has to be driven = BT= 99.86 - 77.13 = 22.73m.

(b) **Distance CD** has to be driven = DT, = 149.31 - 77.13 = 72.18m.

(c) **Length of new roadway** BTT,D= BT + arc TT, +T,D

$$= 22.73 + \left(2\pi \times 66.66 \times \frac{98°20'}{360°}\right) + 72.18$$

= 22.73 + 114.47 + 72.18 = 209.38 m

Example 19. The following data refer to a traverse survey ABCD.

Line	*Bearing*	*Hor. Distance*
AB	N 15°40' E	190.68m
BC	S 75°43' E	159.80m
CD	S 33° 30' W	308.48m

Calculate the co-ordinates of the stations and determine therefrom the area of the enclo sure ABCDA in statute hectares.

Answer : AREA OF A CLOSED TRAVERSE

The co-ordinates of D with respect to origin A -

Line Stn.	Quadrant Bearing θ	Distance in mts. L	Partial Co-ordinates		Total co-ordinates	
			Lat L Cos θ	Dep L sin θ	Latitude	Departure
A	0	0			0	0
AB	N 15°40'E	190.68				
B			+ 183.60	+ 51.49	183.60	+ 51.49
BC	S 75°43'E	159.80				
C			-39.42	+ 154.86	+ 144.18	+206.46
CD	S 33° 30' W	308.48				
D			-256.83	- 170.28	-112.65	+ 36.18

Area of Rectangle

=(183.60 + 112.65)x (51.49+154.86)

= 61131.188 Sq.m.

Area of Triangles :

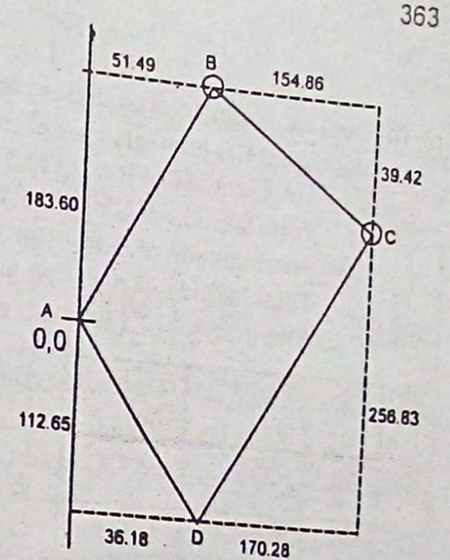
½ (51.49x 183.60) = 4726.782

½ (154.86x39.42) = 3052.290

½ (256.83x170.28) = 21866.506

½ (36.18x112.65) = 2037.830

= 31683.408



Area of enclosure ABCDA = 61131.188 - 31683.042 = 29448.042 Sq.m.

$$\text{Area in Hectares} = \frac{29448.042}{10000} = 2.9448.$$

Example 20. The following are the latitudes and departures of the lines of a closed traverse ABCD :

Line	*Latitude*	*Departure*
AB	- 116.1m	- 44.4m
BC	+ 6.8m	+ 58.2m
CD	+ 80.5m	+ 17.2m
DA	+ 28.8m	- 31.0m

Compute the area of the traverse by (a) co - ordinate method, (b) the departure and total latitudes method and (c) the D.M.D. method.

Answer : AREA OF THE TRAVERSE

(a) **Co - ordinate Method** : Taking the co-ordinates of A as (+200,+200) the results are tabulated below.

Line	Lat (m)	Dep (m)	stn.	Independent Co-ordinates	
				North Lat (y)	East Dep. (X)
			A	200.0	200.0
AB	- 116.1	- 44.4	B	83.9	115.6
BC	+ 6.8	+ 58.2	C	90.7	213.8
CD	+ 80.5	17.2	D	171.2	231.0
DA	+ 28.8	- 31.0	A	200.0	200.0

Area $= \frac{1}{2}\{ y_1 (x_2 - x_4) + y_2 (x_3 - x_1) + y_3 (x_4 - x_2) + y_4 (x_1 - x_3)\}$

$= \frac{1}{2}\{ 200 (83.9 - 171.2) + 155.6 (90.7 - 200) + 213.8 (171.2 - 83.9) + 231 (200 - 90.7)\}$

$= \frac{1}{2}\{ 200 \times (- 87.3) + 155.6 (- 109.3) + 213.8 \times 87.3 + 231 \times 109.3 \}$

$= \frac{1}{2} (- 17,460 - 17,007.08 + 18,664.74 + 25,248.3) = 4722.98$ sq.m

(b) Departure and Total Latitudes Method :

Calculated the total latitudes of the point, starting with as the reference point . Thus

Total latitude of B = - 116.1

Total latitude of C = - 116.1 + 6.8 = - 109.3

Total latitude of D = - 109.3 + 80.5 = - 28.8

Total latitude of A = - 28.8 + 28.8 = 0

The area = ½ Σ (total lat. x algebraic sum of adjoining dep.)

Line	Lat (m)	Dep. (m)	Stn.	Total Lat. (L)	Algeb. Sum of adjoining	Double Area
AB	- 116.1	- 44.4	B	- 116.1	+ 13.8	- 1602.18
BC	+ 6.8	+ 58.2	C	- 109.3	+ 75.4	- 8241.22
CD	+ 80.5	+ 17.2	D	- 28.8	- 13.8	+ 397.44
DA	+ 28.8	- 31.0	A	00	- 75.4	00

Sum (Double Area) = - 9445.96

Since the negative sign does not have any significance,
the actual Area = ½ x9445.96 = 4722.98 sq.m.

(c) **D.M.D. (Double meridian Distance) Method**

Area = ½ Σ ML

Line	Lat. (L)	Dep. (D)	D.M.D. (M)	Product MXL
AB	- 116.1	- 44.4	44.4	- 5154.84
BC	+ 6.8	58.2	92.6	+ 629.68
CD	+ 80.5	+ 17.2	133.6	+ 10754.80
DA	+ 28.8	- 31.0	119.8	+ 3450.24
			Total	9679.88

Area = 1/2 x 9679.88 = 4839.94 sqm.

Exaple 21. Following are the notes of a theodolite traverse in an opencast mine :

Line	Azimuth Bearing	Distance	Vertical Angle
AB	89°54'	350m	Level
BC	150°12'	190m	Level
CD	180°00'	600m	Level
DE	140°18'	155m	- 28°00'
EF	228°36'	800m	- 12°00'

It is proposed to make a haul road connecting A and F. Calculate the length, gradient and not bearing of the haul road following line FA.

Answer : Horizontal length of DE= 155 cos 28° = 136.857m

Horizontal length of EF = 800cos 12° = 782.518m

The co-ordinates of the stations calculated and tabulated below :

Line Stn.	Azimuth	Q.B.	Hor Dist.(m)	partial Co-ordinate		Total Co-ordinate	
				Lat.	Dep.	Lat.	Dep.
A							
AB	89°54'	N 89°54'E	350				
B				+ 0.610	+ 349.999	+ 0.610	+ 349.999
BC	150°12'	S 29°48'E	190				
C				- 164.875	+ 94.425	- 164.265	+ 444.424
CD	180°	Due South	600				
D				- 600.00	-	- 764.265	+ 444.424
DE	140°18'	S39°42'E	155				
E				- 119.257	+ 99.009	- 883.522	+ 543.433
EF	228°36'	S48°36'W	800				
F				- 529.050	- 600.090	-1412.572	- 56.657

Bearing of line FA = $\tan^{-1}\frac{\text{Dep}}{\text{Lat}} = \tan^{-1}\frac{56.657}{1412.572} = 2°17'30"$ NE

Plan length of FA = $\sqrt{\text{Lat}^2 + \text{Dep}^2} = \sqrt{1412.572^2 + 56.657^2} = 1413.704$ m.

In order to calculate the inclined length of the drift, it is necessary to find the difference in level between A and F, as under -

Rise on line DE=155xsin 28°= 72.768m.

Fall on line EF= 800x sin 12° = 166.329m.

∴ Difference in level between A nad F

= 166.329 - 72.768 = 93.561m fall.

Inclination on drift = $\tan^{-1}\frac{93.561}{1412.572} = 3°47'22"$

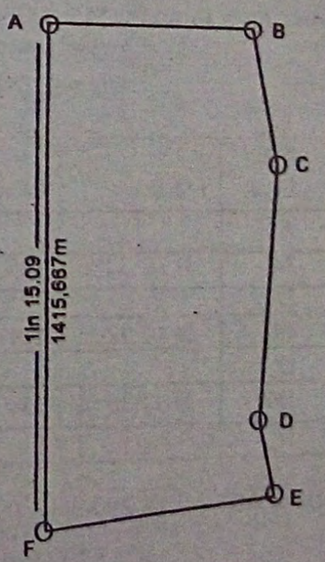
This is equivalent to gradient

$= 1 \text{ in } \frac{1}{\tan 3°47'22"} = 1 \text{ in } 15.09$

Therefore, the drift AF will fall at 1in 15.09 from A to F.

The length of proposed drift (haul road)

$= \frac{1412.572}{\text{Cos } 3°47'22"} = 1415.667\text{m}$



Bearing of drift AF = S 2°17'30" W

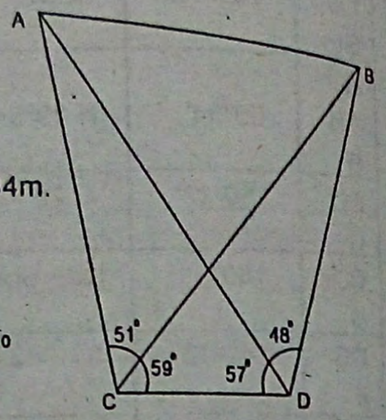
Example 22. It is required to determine the distance between two inaccessible points A and B by observations from two stations C and D, 250m apart. The observed angular measurements from C and D are as follows:

∠ ACB = 51°, ∠ BDA = 48°, ∠ BCD = 59°, and ∠ ADC = 57°.

Calculate the distance of AB

Answer : angle CAD = 180° - (51° + 59° + 57°) = 13°

by Sine Rule, $AC = \frac{CD \times \text{Sin ADC}}{\text{Sin CAD}} = \frac{250 \times \text{Sin } 57^\circ}{\text{Sin } 13^\circ} = 932.059\text{m.}$



In BCD, ∠ CBD = 180° - (59° + 57° + 48°) = 16°

∠ CDB = 57° + 48° = 105°

By Sine Rule , $BC = \frac{CD \times \text{Sin D}}{\text{Sin B}} = \frac{250 \times \text{Sin } 105^\circ}{\text{Sin } 16^\circ} = 876.084\text{m.}$

In triangle ABC, by Cosine Rule,

$AB = \sqrt{AC^2 + BC^2 - 2AC \times BC \times \text{Cos } C}$

$= \sqrt{(932.059)^2 + (876.084)^2 - 2 \times 932.059 \times 876.084 \times \text{Cos } 51^\circ}$

= 780.063 m.

Example 23. The length and bearing of a line AB can not be observed direct in field due to obstacle in traversing. The following data were obtained by observation on the stations C and D of the traverse ABCD. Calculate the length and bearing of AB.

Line	Fore Bearing	Length
CA	258°30'	235m
CD	10°02'	114m
DB	276°30'	454m

Answer : Fore Bearing CA = 258°30' ..... AC Q.B. = N 78°30' E

CD = 10°02' ..... Q.B. = N 10°02' E

DB = 276°30' ..... Q.B. = N 83°30' W

Stn./ Line	Q.B.	Length	Partial Co-ord.		Total Ord.	
			Lat.	Dep.	Lat.	Dep.
A					0.000	0.000
AC	N 78°30' E	235	+ 46.851	+ 230.282		
C					+ 46.851	+ 230.282
CD	N 10°02' E	114	+ 112.257	+ 19.861		
D					+ 159.108	+ 250.143
DB	N 83°30' W	454	+ 51.394	- 451.082		
B					+ 210.502	- 200.939

Length of AB $= \sqrt{(-200.939 - 0)^2 + (+210.502 - 0)^2} = 291.011\text{m.}$

Bearing of AB $= \tan^{-1}\left(\frac{-200.939}{+210.502}\right) = \text{N } 43^\circ 40' 07''\text{ W.}$

Example 24. Calculate latitudes, departures and closing error for the following traverse, and adjust using Bowditch's rule.

Line	*Length (m)*	*Whole Circle Bearing*
AB	89.31	45°10'
BC	219.76	72°05'
CD	151.18	161°52'
DE	159.10	228°43'
EA	232.26	300°42'

Answer : Calculations are given in table below ---

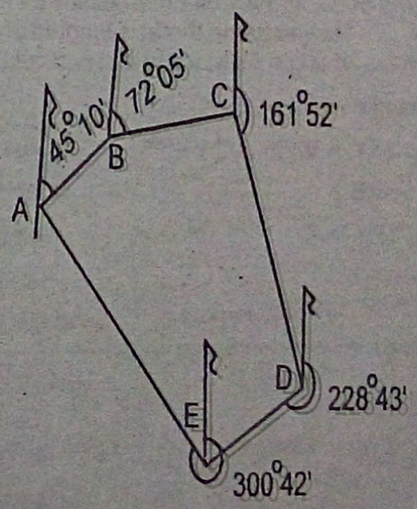
Line	W.C.B.	Length m	Latitude			Departure		
			Calculated	Correction	Corrected	Calculated	Correction	Corrected
AB	45°10'	89.31	+ 62.97	- 0.06	+ 62.91	+ 63.34	- 0.02	+ 63.32
BC	72°05'	219.76	+ 67.61	- 0.13	+ 67.48	+ 209.10	- 0.06	+ 209.04
CD	161°52'	151.18	- 143.67	- 0.09	- 143.76	+ 47.05	- 0.04	+ 47.01
DE	228°43'	159.10	- 104.97	- 0.10	- 105.07	- 119.56	- 0.04	- 119.60
EA	300°42'	232.26	+ 118.58	- 0.14	+ 118.44	- 199.71	- 0.06	- 199.77
	Σ	851.61	+ 0.52	- 0.52	0.00	+ 0.22	- 0.22	0.00

Corrections : For latitude of AB

$= \frac{-0.52}{851.61} \times 89.31 = -0.06\text{m}$

For departure of AB $= \frac{-0.22}{851.61} \times 89.31 = -0.02\text{m}$

Corrections for other lines are determined using the same procedure.



Closing Error = $\sqrt{(0.52)^2 + (0.22)^2} = 0.565\text{m}$

The angle of the error of closure (θ) is given by

$\text{Tan } \theta = \frac{0.22}{0.52} = 0.4231$

$\theta = 22^\circ 56'$

Relative accuracy $= \frac{0.565}{851.61} =$ **1 in 1507**

Example 25. A road is set out on a curve of 300m radius. The total width of the road and verges is 15m. The angle subtended at the centre by the extrimities of the curve is 72 degrees. What is the total area of this curved portion of the road.

Answer : The area required is a sector of an annulus.

Area of annulus = $\pi (R - r) (R + r)$

$= \pi (307.5 - 292.5) (307.5 + 292.5)$

$= \pi \times 15 \times 600 = \pi \times 900$

Area of sector of annulus = $\frac{72}{360}$ x area of annulus.

$= \frac{72}{360} \times \frac{22}{7} \times 900 = 565.72$ Sq.m.

Example 26. The Co-ordinates, in metres, of two triangulation stations A and B are as follows:

A	3950 N	.....	3672. 7 E
B	547 N	.....	2169. 7 W

Observations taken from two traverse stations X and Y to the south of the line BA (X being nearest B) have given the undernoted information :

Angle	AXB	57°56'32"
	AXY	46°51'19"
	BYX	32°12'55"
	BYA	57°56'32"

calculate the co-ordinates of the traverse station X.

Answer : Referring to Figure, let assume a length of 1000m for the length of XY. From this to calculate the length of BX in the triangle BXY, AX in the triangle AXY and AB in the triangle ABX. From the known co-ordinates of A and B the true length AB can be ascertained, following which have a ratio between the assumed arbitrary length of AB and the true length. The true lengths of BX and AX can then easily be obtained by applying the ratio.

Having found the true lengths of AB, BX and AX we solve the triangle ABX and obtain the value of angle ABX, there after calculating the bearing of BX and the co-ordinates of X.

In triangle BXY

$\angle BXY = 57°56'32" + 46°51'19" = 104°47'51"$

$\angle XYB = 32°12'55"$

$\angle YBX = 42°59'14"$

$180°00'00"$

Then $BX = \frac{XY \sin XYB}{\sin YBX} = \frac{1000 \sin 32°12'55"}{\sin 42°59'14"} = 781.85$m.

In triangle AXY

$\angle XYA = 32°12'55" + 57°56'32" = 90°09'27"$

$\angle AXY = 46°51'19"$

$\angle YAX = 42°59'14"$

$180°00'00"$

$AX = \frac{XY \sin XYA}{\sin YAX} = \frac{1000 \sin 90°09'27"}{\sin 42°59'14"} = 1466.61$m.

In triangle ABX, $AB^2 = BX^2 + AX^2 - 2BX.AX.\cos BXA$

Therefore, $AB = \sqrt{BX^2 + AX^2 - 2BX.AX.\cos BXA}$

$= \sqrt{781.85^2 + 1466.61^2 - 2(781.85 \times 1466.61) \cos 57°56'32"}$

$= 1243.0$m.

True Length $AB = \sqrt{(\text{diff. of deps.})^2 + (\text{diff. of lats.})^2}$

$= \sqrt{5842.4^2 + 3403.0^2} = 6761.2$m

Therefore, ratio $= \frac{\text{true}}{\text{assumed}} = \frac{6761.2}{1243.0}$

Thus true length BX is given by $781.5 \times \frac{6761.2}{1243} = 4252.9$

In triangle ABX,

Semi perimeter = ½ (6761.2 + 4252.9 + 7977.6) = 9495.9 m.

$\sin \frac{B}{2} = \sqrt{\frac{(s - a)(s - x)}{ax}}$

$= \sqrt{\frac{(9495.9 - 4252.9)(9495.9 - 6761.2)}{(4252.9 \times 6761.2)}}$

Therefore $\frac{B}{2} = 44°55'16"$ and $B = 89°50'32"$

Bearing $BA = \tan^{-1} N \frac{5842.4}{3403.0} E = N\ 59°46'48"\ E$

Azimuth BA = 59°46'48"

Add angle ABX = 89°50'32"

149°37'20" = azimuth of BX

and S 30°22'40" E = bearing of BX

Latitude of X with B as zero = 4252.9 cos 30°22'40"

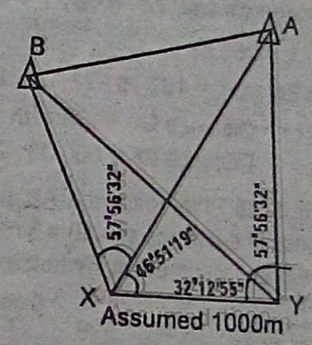
= South 3669.3m.

Departure of X with B as zero = 4252.9 sin 30°22'40"

= East 2150.7m.

Therefore, **Reduced latitude** of X = 547 - 3669.3 = 3122.8m

Reduced departure of X = - 2169.7 + 2150.7 = - 19.0m.



Example 27. From a common point A, traverse are conducted on either side of a harbour os follows :

Line	Length	Bearing
	Traverse 1	
AB	200m	85°26'20"
BC	100m	125°10'40"
	Traverse 2	
AD	225m	175°50'00"
DE	500m	85°06'40"

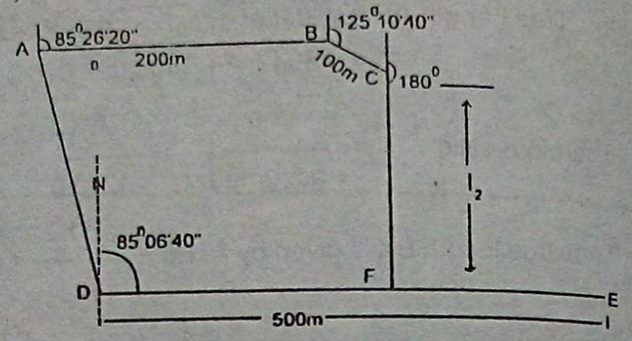
Calculate the distance from C to a point F on DE due south of C and the distance EF.

Answer: Traverse ABCFDA is closed. Let the length DF and CF be respectively l_1 and l_2

Σ Latitude = 0

or 200 cos 85°26"20' + 100 cos 125°10'40" - l_2 + l_1 cos 265°06'40" + 225cos 355°50'00" = 0

or, 15.90 + (-57.61) - l_2 + l_1 (-0.085) + 224.41.0



or, 0.085 l_1 + l_2 = 182.70 .....(a)

Σ Departure = 0

or, 200 sin 85°26'20" + 100 sin 125°10'40" + 0 + l_1 sin 265°06'40" + 225 sin 355° 50'00" = 0

or, 199.37 + 81.74 - 0.996 l_1 - 16.35 = 0 ..... (b)

or, l_1 = 265.82

From Eq. (a) l_2 = 182.70 - 22.59 = 160.11

Distance CF = 160.11m

Distance EF = 500.00 - 265.82 = 234.18m.

Example 28 : Two roadways AB and CD are to be joined by a curved road of 200 m radius. The bearings of the roads are AB = 31°30' and CD = 113°24'

AB and CD meet at T when extended. The co-ordinates of B, D, and T in metre as follows :

	X	Y
B	125.30	168.95
D	72.26	330.47
T	375.57	499.06

How far must the roads be extended to meet the tangent points ? What will be length of curved road from B to D ?

Answer : Let AB and CD be the two roadways and E and F be the tangent points for the curve having a radius OE = 200m.

Bearing of AB = 31°30'

Bearing of CD = 113°24'

$\angle DTB = 113°24' - 31°30' = 81°54'$

Tangent distance, ET $= FT = R\cot\frac{81054'}{2}$

$= 200 \times \cot 40°57' = 230.48$ m.

Length of curved road EF $= 200 \times 2\pi \frac{180° - 81°54'}{360°} = 200 \times 2 \times \frac{22}{7} \times \frac{98°06'}{360°} = 342.57$m

Also $BT = \sqrt{(375.57 - 125.30)^2 + (499.06 - 168.95)^2}$

$= \sqrt{250.27^2 + 330.11^2} = 414.25$m.

$BF = BT - FT = 414.25 - 230.48 = 183.77$ m.

$DT = \sqrt{(375.57 - 72.26)^2 + (499.06 - 330.47)^2}$

$= \sqrt{303.31^2 + 168.59^2} = 347.02$m.

Now DE = DT - ET = 347.02 - 230.48 = 116.54 m.

The roadways **AB and CD must be driven 183.77m. and 116.54m respectivelty** to meet the tangent points.

Length of curved road EF = 342.57m.

Length of roadway DEFB = DE + EF + FB

= 116.54 + 342.57 + 183.77 = **642.88 m.**

Example 29. In order to correlate the surveys of a mine with the National Grid, the following observations were taken----

Two Ordnance Survey Station X and Y are available and have the undernoted grid reference , the values being expressed in metres from the origin :

Tower X ..... Easting 399468.32 Northing 698852.48

Pillar Y..... " 400130.83 " 701887.75

As these station are not visible from the mine base line in the vicinity of the shaft,a station A was stablished to the east of X and Y, and from this station a traverse was made to the centre of the shaft D.

The value of the angles taken at Y and A are ----

Angle AYX, 50°17'42" ; angle XAY, 38°22'15" ; angle XAB, 242°47'33".

From station A the traverse was made as under:

Line	Hor. angle	Distance in Feet
AB	----	1754.5
BC	150°17'30"	576.4
CD	246°51'34"	386.0

The above distances have been corrected for the slope of the ground.

From the above data calculate the National Grid co-ordinates of D in the metres to one place of decimals and the Grid bearing of the colliery base line CD.

In converting feet to metres use the value 1 metre = 3.28084 feet. Considering that the Easting of the co-ordinates are in the neighbourhood of 400,000 and that the height above Ordnance Datum of A to D is 500 to 400 feet,state,without calculating, what further adjustments would require to be made in order to obtain the fully corrected National Grid co-ordinate of D.

Answer: The relative positions of the tower X, the pillar Y, and the traverse stations A , B , C and D are indicated by the figure.

In the triangle AYX, the angle X = 180° - A -Y = 180° - 38°22'15" - 50°17'42" = 91°20'03"

Bearing of XY = N $\tan^{-1}\frac{\text{Diff. in Dep.}}{\text{Diff. in Lat}}$ E = N $\tan^{-1}\frac{671.51}{3035.27}$ E = N 12°28'28" E

Length of XY = 3035.27 Sec. 12°28'28" = 3108.76 m.

In triangle XAY, $AX = \frac{2108.76 \text{ Sin } 50°17'42"}{\text{Sin } 38°22'15"} = 3852.76$ m.

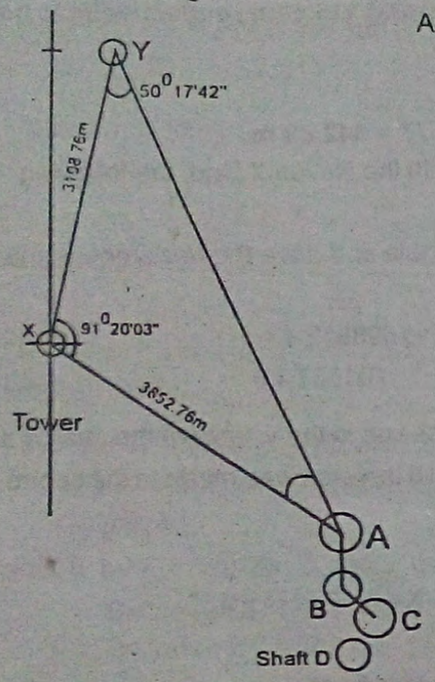
To reduce traverse distance in feet to metres.

AB = 1754.5 ÷ 3.28084 = 534.77m

BC = 576.4 ÷ 3.28084 = 175.68m

CD = 386.0 ÷ 3.28084 = 117.65m

Calculation of bearings of the lines in the traverse -----



Azimuth of line XY = 12°28'28", and Q.B. = N 12°28'28" E

91°20'03"

XA = 103°48'31" and Q.B. = S 76°11'20" E

+ 180°00'00"

+ 242°47'33"

AB = 526°36'04" and Q.B. = S 13°23'56" E

- 360°00'00"

166°36'04"

+ 180°00'00"

+150°17'30"

BC = 496°53'34" and Q.B.= S 43°06'26" E

- 360°00'00"

136°53'34"

+ 180°0'00"

+ 246°51'34"

563°45'08"

- 360°00'00"

C D = 203°45'08" and Q.B = S 23°45'08" W

The co-ordinates of the in the traverse in their relation to the Grid Co-ordinates of X are given in the following ---

Line	Beraing	Distance (m)	Departure		Latitude	
			E	W	N	S
Stn X			399468.32		698852.48	
XA	S 76°11'29" E	3852.76	5741.15			920.63
AB	S 13°23'56" E	534.77	123.64			520.28
BC	S 43°06'26" E	175.68	120.02			128.29
CD	S 23°45'08" W	117.65		47.38		107.69
			403453.13	47.38		1676.89
Grid Co-ordinates of D =			403405.75 E		697175.59 N	

The Grid bearing of the CD is as given in the above table. In projecting the Grid co-ordinates into the mine plan the further adjustments necessary are : (1) As projected lengths are less than actual distances according to local scale factors, adjustments must be made for local scale factor before actual distances are plotted on the plan. (2) Distances must be reduced to ordnance datum before plotting.

Example 30 : The positions of two railway lines are fixed with reference to an assumed point of origin. One line passes through the point (1000, 3000) in a direction N 70° W and the other through the point (2500 , 800) in a direction N 10° W , the distances being measured in metre. Find the positions of the contact points of a circular curve to join the lines, the radius of the curve being 400m.

Answer : In the Figure, there are two possible curves. It is proposed to calculate for the position shown and to use the method of co-ordinate geometry.

Graient of first line = - tan20° = - 0.364

General equation is y = ax + b

Substituting the values given in the question

3000 = - 0.364 x 1000 + b

b = 3364

Thereby y = - 0.364 + 3364 for first line.

Gradient of second line = - tan 80° = - 5.6713

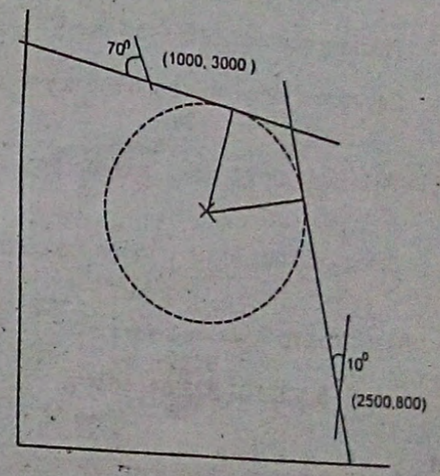
y = ax + b

800 = - 5.6713 x 2500 + b

b = 800 + 14178 = 14978

y = - 5.6713x + 14978 for second line.

Now the centre of the circle will be at the intersection of two lines parallel respectively to each of the given lines and distant 400m from them.



Equation of the two parallel will be :

$y = - 0.364x + 3364 - 400\sqrt{(-0.364)^2 + 1}$

$= - 0.364x + 2938.4$

and $y = - 5.6713x + 14978 - 400\sqrt{(-5.6713)^2 + 1}$

$= - 5.6713x + 12675$

The intersection of the two lines is the centre of the circle solving the two as simultaneous equations we have :

$- 0.364x + 2938.4 = -5.6713x + 12675$

$5.6713x - 0.364x = 12675 - 2938.4$

$5.3.73 = 9736.6$

$X = 1834$

Therefore $y = - 0.364x1834+2938.4$

$= - 668 + 2938.4 = 2770.4$

Centre of circle is at position (1834, 2770)

Equation of original first line is y = - 0.364x + 3364

Therefore equation of radius to contact point is $y = \frac{x}{0.364} + b$

Since the line passes through the centre of the circle, then

$$2270 = \frac{1834}{0.364} + b$$

$$b = 2270 - 5040 = -2770$$

Therfore equation of this radius is $y = \frac{x}{0.364} - 2770$

Equation of original second line is $y = -5.6713x + 14978$

Thus equation of radius to contact point is $y = \frac{x}{5.6713} x + b$

Since this line passes through the centre of the circle, then

$$2270 = \frac{1834}{5.6713} + b$$

$$b = 2270 - 323.4 = 1946.6$$

Therefore equation of this radius is $y = \frac{x}{5.6713} + 1946.6$

To find the position of the contact point on the first line we use the equations of that line and the intersecting radius :

$$y = -0.364x + 3364$$

$$y = \frac{x}{0.364} - 2770$$

Therefore $\frac{x}{0.364} + 0.364x = 3364 + 2270$

$$3.112x = 5634$$

$$x = 1810$$

Therefore $y = -0.364 \times 1810 + 3364 = 2705$

To find the second contact point :

$y = -5.6713x + 14978$

$$y = \frac{x}{5.6713} + 1946.6 = 0.1763x + 1946.6$$

$$0.1763x + 5.6713x = 14978 - 1946.6$$

$$5.8476x = 13031.4$$

$$x = 2228$$

$y = -5.6713 \times 2228 + 14978 = 2342$

Therefore the tangent points of the lines to the curve are (1810,2705) and (2228, 2342) or, in other words, at the positions N 1810, E 2705 and N 2228, E 2342.

Knowing two points on the curve and its centre, it would be possible to obtain the equation of the curve.

Example 31. A straight tunnel is to run between two points A and B whose co-ordinates are as under.

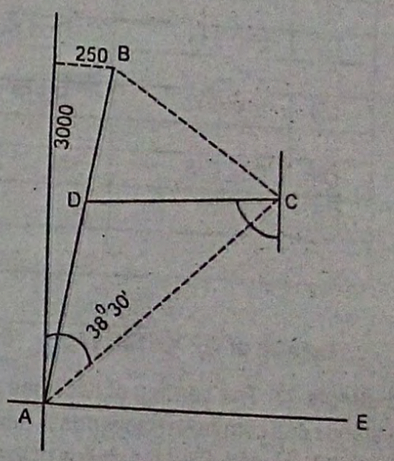
Point	Northing	Easting
A	0.0	0.0
B	3000m	250m

A shaft is to be sunk at a point D which is at the middle of AB. As it is impossible to measure along AB directly, the point D is to be fixed from another point C whose co-ordinates are 1760m northing and 1395 m easting.

Calculate : (a) the co-ordinates of D, (b) the length and bearing of CD, and (c) the aangle ACD, if the bearing of CD, and (c) the angle ACD, if the bearing of AC is 38°30' east of north.

Answer : (a) D is the mid - point of AB, its co-ordinates are

$N = \frac{1}{2}(3000 + 0) = 1500m$, $E \frac{1}{2}(250 + 0)\ 125m.$



(b) From the co-ordinates of C and D,

Length of $CD = \sqrt{(1760 - 1500)^2 + (1395 - 125)^2}$

$= \sqrt{260^2 + 1270^2} = 1296.34m$

$\text{Tan } \theta = \frac{1270}{260} = 4.8846 \quad \therefore \theta = 78°26'$

The line CD lies in the third quadrant with respect to the origin at C.

(c) The whole circle bearing of

$CD = 180° + 78°26' = 258°26'$

$RB = S\ 78°26'\ W$

$\angle ACD = 78°26' - 38°30' = 39°56'$

Example 32: The distance between two in accessible points P and Q is to be found. For which the following angular observations were made from the stations C and D 500m apart.

$\angle PCD = 60°$, $\angle PCQ = 45°$, $\angle QDC = 40°$, $\angle QDP = 50°$

Calculate the distance PQ.

Answer : The sides CD, DP, PQ and QC from a closed traverse.

To find the lengths DP and CQ --

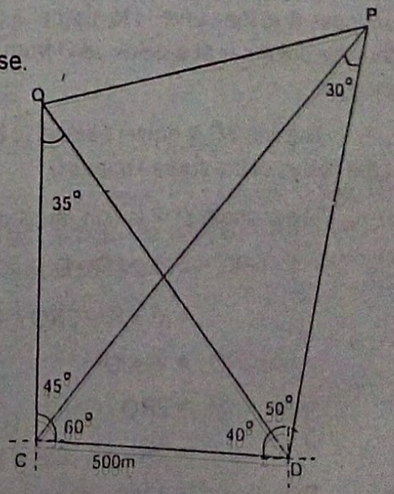
In triangle CDQ, $\angle CQD = 180° - 105° - 40° = 35°$

By Sine Rule , we have

$$\frac{CQ}{\text{Sin } 40°} = \frac{500}{\text{Sin } 35°}$$

$$CQ = \frac{500 \times \text{Sin } 40°}{\text{Sin } 35°} = 560.333\ m.$$

In triangle CDP, $\angle CPD = 180° - 60° - 90° = 30°$



By Sine Rule, we have,

$$\frac{DP}{\text{Sin } 60^0} = \frac{CD}{\text{Sin } 30^0} \quad \text{or} \quad DP = \frac{500 \times \text{Sine } 60^0}{\text{Sin } 30^0} = 866.025 \text{ m.}$$

Considering CQ as a reference meridian, the bearing of CQ is therefore ' zero '.

Bearing CQ = 0° Due north .

Bearing CD = S (180° - 105°) = S 75° E .

Bearing DP = (105° + 180° + 90°) - 360° = N 15° E

Line Stn	Length (m) L	Reduced Bearing (θ)	Partial Co-ordinates	
			Latitude (L cos θ)	Departure (L sin θ)
Q			0	0
QC	560.333	Due South		
C			- 560.333	0
CD	500.000	S 75° E		
D			- 129.409	+ 482.963
DP	866.025	N 15° E		
P			+ 836.492	+ 224.137
			+ 146.750	+ 707.100

Length of QP = $\sqrt{146.750^2 + 707.100^2}$ = 722.165 m.

Example 33. The bearing of two lines AB and BC are N 36°36' E. and N 80°00' E. respectively. At a distance of 276m from B towards A and 88m at right angles and to the East of the line AB a station Y has been located. Find the radius of the curve to pass through the point Y and also to be tangential to the two lines.

Answer : As a tangent to a curve at any point on the curve is a straight line, it is hardly right to state that the curve is to tangential to the lines AB and BC.

In proceeding to the solution of this very interesting problem in algebraic geometry we may suppose that the curve is to start at a point T at a distance x measured back along the line AB, and that the point Y is at a distance O from the point A, the line AY being at right-angles to TA, as shown in figure.

The line YD is drawn parallel to the line AT, hence AY equals TD and the distance DP equals R (the radius of the curve) minus 0.

The triangle YDP is right-angled at D, and :--

$$R^2 = x^2 + (R - O)^2$$

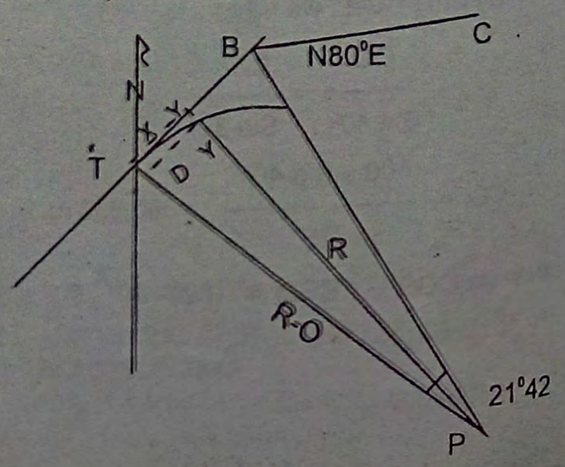
$$= x^2 + R^2 - 2RO + O^2$$

hence $2RO = x^2 + O^2$

and $x^2 = 2RO - O^2$

$$= 2R \times 88 - 88^2 = 176R - 7744$$

Therefore $x^2 = 176R - 7744$



As this equation contains two unknown quantities, x and R, it is necessary that we should find another equation having the same unknowns in order that the unknowns may be found by the rules of algebra relating to simultaneous and quadratic equations.

Since the bearing of the line BC is N 80°00' E. and the bearing of AB is N 36°36' E., the angle of deflection equals 80°00' - 36°36' = 43°24' and the angle BPT is half the angle at the centre of the curve, viz. : 21°42' Now x + 276 = R tan 21°42' = 0.3979 R.

We have now abtained two equations involving x and R,

$$x^2 + 7744 = 176R \quad \text{—} \quad (1)$$

and $x + 276 = 0.3979R$ — (2)

Multiplying (1) by 0.3979 and (2) by 176 we get :---

$$176x + 48576 = 69.53R \quad \text{—} \quad (3)$$

$$0.3979x^2 + 3081.34 = 69..53R \quad \text{—} \quad (4)$$

$$0.3979x^2 + 3081.34 = 176x + 48576 \quad \text{—} \quad (5)$$

Dividing by 0.3979 we obtain a quadratic equation in x :--

$$x^2 + 7744 = 443x + 122081$$

$$x^2 - 443x = 114{,}337$$

Completing the square :--

$$x^2 - 443x + 221.5^2 = 114337 + 49062$$

$$(x - 221.5)^2 = \pm \sqrt{163399}$$

$$x - 221.5 = \pm 404.2$$

and the positive root $x = 404.2 + 221.5 = 625.7$

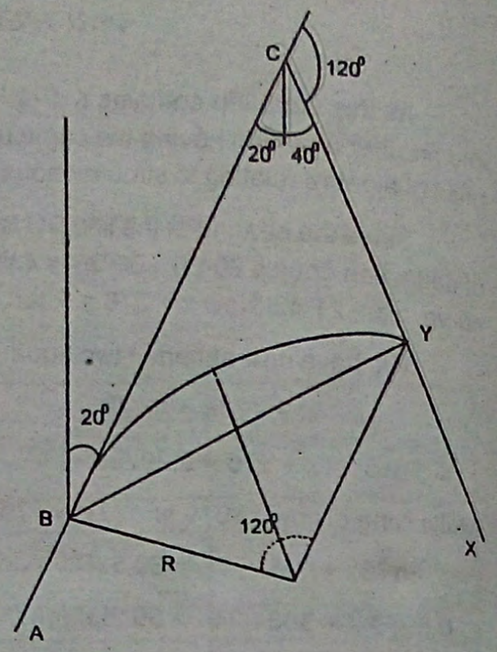
Now, the radius of the curve, starting at T. and passing through the point Y, is found by substituting x = 625.7 in equation (2) thus :---

$$625\ 7 + 276 = 0.3979R$$

and R = 901.7 ÷ 0.3979 = 2,391m.

Example 34: Two surface roads of an opencast mine are to be connected by a smooth curve. The directions and co-ordinates of the road turning points are as follows. The direction of the road AB in N 20° E and roadway XY is N 40° W. The co-ordinates of the tangent point B = 3245 Lat. and 1565 Dep. The co-ordinates of the tangent point Y = 3310 Lat. and 1670 Dep. Find out the radius of the curve.

Answer: A rough sketch is drawn below ----



Radius of the curve = R

Long chord = BY

Co-ordinates of B : 3245 Lat.

1565 Dep.

Co-ordinates of Y : 3310 Lat

1670 Dep.

Length of the long chord by

$= \sqrt{(\text{Dep. of Y - Dep. of B})^2 + (\text{Lat of Y - Lat of B})^2}$

$= \sqrt{(1670 - 1565)^2 + (3310 - 3245)^2} = 123.491\text{m}.$

Angle YCB = $20^0 + 40^0 + 60^0$

Angle at centre = $180^0 - 60^0 = 120^0 = \theta$.

Length of long chord = $2\ R \sin \frac{\theta}{2} = 2\ R \sin 60^0 = 123.491$

$$R = \frac{123.491}{2 \sin 60^0} = 71.291$$

Radius of the curve = 71.291m.

Example 35. The co- ordinates of the centres of two shafts A and B are respectively as follows :

	Latitude	*Departure*
Shaft A	N 658.5m	W 233.7m
Shaft B	N 108.0m	E 246.3m

The depths of the shafts and the levels at the surface are respectively.

	Depth	*Surface level*
Shaft A	180m	246m above datum
Shaft B	135m	160.8m above datum

Determine the bearing, length and gradient of a cross-measures drift to connect the bottom of shaft A to the bottom of shaft B.

Answer : To determine the plan length and bearing of the drift:

	Latitude	*Departure*
Shaft A	N 658.m	W 233.7m
Shaft B	N 108.0m	E 246.3m
Difference	550.5m	480.0m

Plan length = $\sqrt{550.5^2\text{m} + 480.0^2}$ = 730.37679m

Bearing of AB = $\tan^{-1} \frac{480}{550.5}$ = S 41°05'11" E

To determine the reduced levels of the bases of the shafts :

	A	B
Surface level	+ 246m	+.160.8m
Depth of shafts	180m	135.0m
R.L. of shaft base	+ 66m	+ 25.8m

Difference in level A to B = 66 - 25.8 = 40.2 (-)

Gradient of AB = 40.2m in 730.377 = **1 in 18.17 dip.**

Inclined length of AB = $730.377 \times \frac{\sqrt{18.17^2+1}}{18.17}$ = **731.48m**

Example 36. The latitudes and departures of the lines of a closed traverse are given below. Calculate the area of the traverse.

Line	Northing	Southing	Easting	Westing
AB		157.2	154.8	
BC	210.5		52.5	
CD	175.4			98.3
DA		228.7		109.0

Answer : The area is calculated using the D.M.D. method. The reference meridian is taken through A :

Line	Lat (L)	Dep	DMD (M)	Product MxL
AB	- 157.2	+ 154.8	154.8	- 24334.56
BC	+ 210.5	+ 52.5	362.1	+ 76222.05
CD	+ 175.4	- 98.3	316.3	+ 55479.02
DA	- 228.7	- 109.0	109.0	- 24928.30

2x Area = Σ 82438.21

Area = $\frac{82438.21}{2}$ = 41219.105m.

Example 37. Calculate the maximum speed in kilometres per hour allowed on a curve of 90 metres radius, and the distance between the centres of the rail is 1.225 metres, and the elevation must not exceed 4.5 cms.

Answer : Super - elevation or cant in cms = h = Gauge in cms x $\frac{V^2}{gR}$

Given, Gauge of rails = 1.225m = 122.5cms.

h = 4.5cms

R = 90m

h = Guage in cms X $\frac{V^2}{gR}$; or $V^2 = \frac{h \times gR}{\text{Gauge}} = \frac{4.5 \times 9.8 \times 90}{122.5} = 32.43$

Maximum speed of train = $\sqrt{32.43}$ = 5.7 metre per second.

$$= \frac{5.7 \times 60 \times 60}{1000} \text{ Km/hour} = \textbf{20.52 Km/ hour.}$$

Example 38. A straight line OQ of length 2000m is required to be set out at right angles to a given line OP. This is done by traversing from O towards Q. The observations are as follows :

Line	*Length*	*Bearing*
OP	-	360°00'
OR	731m	113°48'
RS	467m	81°18'
ST	583m	105°57'

Compute the necessary length and bearing TQ.

Answer : Let the latitude and departure of TQ be L_1 and D_1 respectively.

Line	Length	Bearing	Latitude	Departure
OR	731m	113°48'	- 294.99	+ 668.84
RS	467m	81°18'	+ 70.64	+ 461.63
ST	583m	105°57'	- 160.21	+ 560.56
TQ	?	?	L_1	D_1
QO	2000m	270°00'	0.00	- 2000.00
		Σ	$- 384.56 + L_1$	$- 308.97 + D_1$

Σ Latitude = 0

or, $- 384.56 + L_1 = 0$ Or, $L_1 = 384.56$m

Σ Departure = 0

or, $- 308.97 + D_1$ Or, $D_1 = 308.97$m

Length of TQ

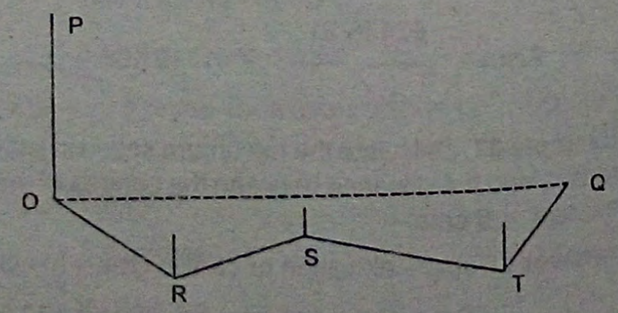
$\sqrt{L_1^2 + D_1^2} = \sqrt{384.56^2 + 308.97^2}$

= 493.30m.

$$\tan \theta = \frac{308.97}{384.56} = 0.8034$$

$\theta = 38°47'$

Quadrantal Bearing of TQ = N 38°47' E



Example 39. A straight tunnel is to run between two points A and B co-ordinates are given below :

Points	*Co - ordinates*	
A	0	0
B	3014 N	254 E
C	1764 N	1398 E

It is desired to sink a shaft at D, the middle point AB, but it is impossible to measure along AB directly, so D is to be fixed from C, a third known point.

Calculate : (a) Co - ordinates of D, (b) Length and bearing of CD, (c) Angle ACD, given the bearing of AC as 38°24' East of North.

Answer : (a) Co - ordinates of D:

Latitude of D = (3014 - 0) ÷ 2 = 1507 N

Departure of D = (254 - 0) ÷ 2 = 127 E

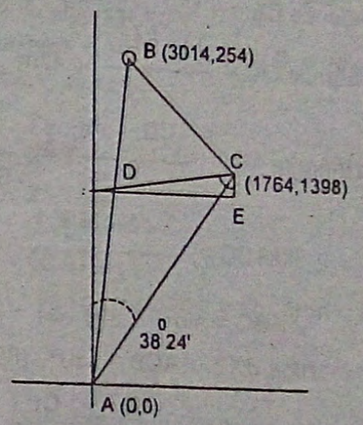
(b) **Length of Bearing CD** :

CE = 1764 - 1507 = 257m

DE = 1398 - 127 = 1271m

∴ Length of CD = $\sqrt{DE^2 + CE^2}$

$= \sqrt{1271^2 + 257^2} = 1296.723$m



Bearing of CD $= \tan^{-1} \frac{\text{difference of deps}}{\text{difference of lats}} = \tan^{-1} \frac{1271}{257} =$ **S78°34'W**

(c) **Angle ACD** = ∠ ECD - ∠ ECA = 78°34' - 38°24' = 40°10'

Example 40. Two parallel roads AB and CD are to be connected for locomotive haulage by reverse curves of equal rodii commencing at B and ending at C.

The local co - ordinates of the points are follows :

A	E 129.06m	N 150.03m
B	E 240.27m	N 201.18m
C	E 151.29m	N 102.03m

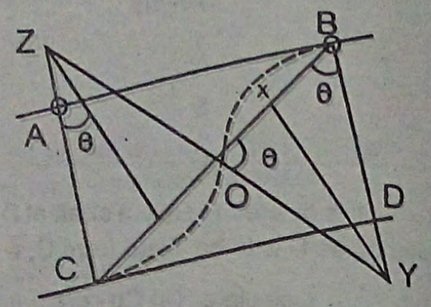
Calculate the radius of the curves to meet these requirements.

Answer : In figure CCB is the required curve with equal radius CZ and BY. From given co-ordinates:

$$\text{Bearing of AB} = \tan^{-1} \frac{240.27 - 129.06}{201.18 - 150.03} = \tan^{-1} \frac{111.21}{51.15} = \text{N } 65°18' \text{ E}$$

$$\text{Bearing of AC} = \tan^{-1} \frac{151.29 - 129.06}{102.03 - 150.03} = \text{S } 24°51' \text{ E}$$

Therefore, $\angle BAC = 180^0 - (65^0 18' + 24^0 51') = 89^0 51'$



Distance $AB = \sqrt{\text{diff. of dep}^2 + \text{diff. of lat}^2}$

$= \sqrt{111.21^2 + 51.15^2} = 122m.$

Distance $AC = \sqrt{22.23^2 + 48^2} = 52.89m$

In triangle BAC,

AB = 122m, AC = 52.89m

and $\angle BAC = 89^0 51'$

Distance $CB = \sqrt{122^2 + 59.89^2 - 2 \times 122 \times 52.89 \cos 89^0.51'} = 132.84m.$

Distance $OC = \frac{132.84}{2} 66.42m$

In triangle ABC, $\frac{CB}{\text{Sin } 89^0 51'} = \frac{AC}{\text{Sin ABC}}$

$$\text{Sin ABC} = \frac{AC \times \text{Sin}89^0 51'}{CB} = \frac{52.89 \times 0.9999}{132.84} = 0.398144$$

$\angle ABC = \sin^{-1} 0.398144 = 23^0 27'43"$

In trianlge BXY, $\angle ACB = 180^0 - (89^0 51' + 23^0 27'43") = 66^0 41'17"$

$\angle ACB = \angle CBY$, and $\angle OBY = \angle BOY$

$\therefore$ OY = BY = radius of the curve. Point X is mid - point of OB,

or, OX = XB and $\angle OXY = 90^0$

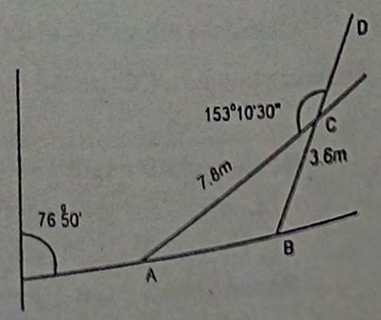
$OX = 66.420 \div 2 = 33\ 21m$

$$\frac{OX}{OY} = \text{Cos } 0 \qquad \therefore OY = \frac{OX}{\text{Cos } 0} = \frac{33.21}{\text{Cos } 66^0 41'17"} = 83.91m.$$

Radius of required curves is 83.91metres.

Example 41 : In the weisbach triangle ABC, a and B are two plumb lines in a vertical shaft, the vertical plane containing the plumb lines having an azimuth of 76°50'00" clockwise from North. C is a theodolite station in an inset and AC and BC measure 7.8m and 3.6m respectively. The angle BCA is 00°12'30" and the ACD measured clockwise from CA is 153° 10'30". Find the azimuth of CD.

Answer : The figure illustrate the question.



Given, AC = 7.8m

BC = 3.6m

$\angle ACB = 12'30"$, and $\angle ACD = 153^0 10'30"$

By Cosine Rule, $AB = \sqrt{AC^2 + BC^2 - 2AC.BC \times \text{Cos } 12'30"}$

$= \sqrt{7.8^2 + 3.6^2 - 2 \times 7.8 \times 3.6 \times \text{Cos } 12'30"} = 4.20m.$

By sine Rule, $\frac{BC}{\text{Sin CAB}} = \frac{AB}{\text{sin } 12'30"}$; or, $\frac{3.6}{\text{Sin CAB}} = \frac{4.20}{\text{Sin } 12'30"}$

Since the angles are very small

$$\text{Angle CAB} = \frac{3.6}{4.2} \times 750 \text{ seconds} = 642 \text{ seconds.}$$

$\therefore$ CAB = 10'42".

Azimuth of AC = 76°50' - 10°42' = 76°39'18"

Azimuth of CD = 76°39'18" - (180° - 153°10'30")

= 76°39'18" - 26°49'30" = 49°49'48"

Example 42. The correlation of the surface and underground surveys of a colliery was based on two wires, A and B, in No. 1 shaft, the surface bearing of which was 116°02'.

Later it was possible to check the correlation by a traverse from A to a wire C in No. 2 shaft. The underground traverse from A to C based on AB was under :

		Azimuth	*Length*
from Wire A:	Line 1	116°02'	4.41m
	Line 2	92°24'	25.74m
	Line 3	35°01'	92.57m
	Line 4	83°12'	50.01m

The surface bearing AC was found to be 59°18'. What was the error in the underground bearing of AB ?

Answer : The azimuth of the line AB at surface, joining two wires A and B, is 116°02'.

If the underground bearing of the closing line, AC, be calculated, it may be compared with the given surface bearing of 59°15' and the difference between the two bearings will be the error in the underground beraing of AB. Such error may arise owing to two plub-lines not being precisely vertical.

The co-ordinates are set out as follows :

LIne	azimuth	Length m	Partial co - ordinates		Remarks
			lat.	dep.	
1	116°02'	4.41	- 1.09355	+ 3.9625	
2	92°24'	25.74	- 1.0778	+ 25.7174	
3.	35°01'	92.57	+ 75.8134	+ 53.1180	
4.	83°12'	50.01	+ 5.9214	+ 49.6582	
			+ 78.7215	+132.4561	

Underground bearing of $AC = N \tan^{-1} \frac{132.4561}{78.7215} E = N\ 59^0 16'34"E$

Surface bearing of AC = 59°18'

Error in underground bearing of AB = 59°18' - 59°16'34"

= 00°01'26" less

Example 43. Given the following total co -ordinates of the stations at the corners of a five sided enclosure, find the area of the field in hectares.

Station	*Latitude*	*Departure*
A	N65m	E 37.5m
B	S 47.5m	E 52.0m
C	S 24.0m	W 60.0m
D	N 33.0m	W 10.0m
E	N 60.0m	W 45.0m

Answer : The area can be found by applying the double area method, the rule of which states that : The area is equal to one half the algebraic sum of the products obtained by multiplying the total departure of each station by the difference in total latitude of adjacent stations.

Thus, starting at station, then to B, to C, etc., we have :

Stn 1	Total Lat 2	Total Dep 3	Diff.in Lat off adj.stns.	Double Area (Col. 3 x 4) +	-
A	+ 65.0	37.5	(60) - (- 47.5) = 107.5	4031.25	
B	- 47.5	+ 52.0	(65) - (-24) = 89.0	4628.00	
C	- 24.0	- 60.0	(- 47.5) - (33) = 80.5	4830.00	
D	+ 33.0	- 10.0	(- 24) - (60) = - 84.0	840.0	
E	+ 60.0	- 45.0	(33 - (65) = - 32.0	1440.0	

Double Area = 15769.25 sq.m.

Area of the enclosure, ABCDEA = 15769.25 ÷2 = 7884.625 sq.m.

$$= \frac{7884.625}{100 \times 100} \text{ Hectares.}$$

= 0.7884625 Hectares

Example 44. The boundaries of a colliery leasehold have been defined by the National Grid co - ordinates of four point A, B, C and D, the area being bounded by straight lines joining these points. For convenients the full grid co - ordinates have been reduced to an origin X and co -ordinates in relation to X are as stated below :

Point	Easting	Northing
A	0	1250
B	910	3850
C	6170	4305
D	4880	0

Calculate from the co -ordinates, wjthout plotting or scaling, the area of the leasehold in square metres. Thereafter convert this area into hectares and also in acres.

Answer : When the co - ordinates of the corners of a closed polygon is as follows -

Multiply the sum of adjacent total latitudes by the difference of adjacent total departures and divide the algebraic sum of the products by two.

Calculation :

Stn.	Total Lat.	Total Dep.	Sum of Adj. Lats	Differ. of Adj. deps.	Double +	Areas -
A	1250	0	5110	910	4641000	
B	3850	910	8155	5260	42895300	
C	4305	6170	4305	- 1290		5553450
D	0	4880	1250	- 4880		6100000
					47536300	11653450
					11653450	

Algebraic Sum = 35882850

The area of the leasehold = 35882850 ÷ 2 = 17941425 sq.m.

= 1794.1425 hects.

0.4047 hectares = 1 acre

∴ 1794.1425 hectares = 1794.1425 ÷ 0.4047 = 4433.265 acres.

Example 45. A surface borehole of 100m depth has been drilled at an inclination of 60° in due north direction from surface.

Results obtained by borrehole survey are given below :

Sl. No. (Line)	Length of the hole	Inclination	Azimuth
1	0.00m	60°	Dee North
2	50.00m	56°	N 10° E
3	100.00 m	50°	N 15° E

Calculcate the latitude and departure of the bottom position of the borehole. Also find out the length and bearing of the deviation of the borehole between surface and bottom points.

Answer : Horizontal length 1- 2 = 50 x Cos 56° = 27.960m.

Horizontal length 2 - 3 = (100 - 50) Cos50° = 32.139m.

Stn. Line.	Horizontal Length	Q.B.	Partial Co - ordinates Latitude	Departure	Total Co - ordinates Latitude	Departure
1					0	o
1 - 2	27.960	N 10°E				
2			+ 27.539	+ 4.855	+ 27.539	+ 4.855
2 - 3	32.139	N 15°E				
3			+ 31.044	+ 8.318	+ 58.579	+ 13.137

Latitude at bottom of borehole = 58.579m.

Departure at bottom of borehole = 13.137m.

Length of deviation of borehole from surface to bottom points

$$= \sqrt{58.579^2 + 13.137^2} = 60.034\text{m.}$$

$$\text{Bearing of deviation} = \tan^{-1}\frac{\text{Dep}}{\text{Lat}} = \tan^{-1}\frac{13.137}{58.579} = \text{N}12°38'24"\text{E.}$$

Example 46. The haulage roadways from two districts of a flat seam of a colliery are to be driven along DC and BC to join a main locomotive haulage roadway at C. The roadway DC and BC are to be 375m and 318m lengths respectvely. The co - ordinates of the traverse connecting B and D are as follows :

No	*Line*	*Length*	*Bearing*	*Latitude*	*Departure*
1	AB	217.71	S59°45'E	- 109.676	+ 188.065
2	BC	318.00	?	?	?
3	CD	375.00	?		
4	DE	283.50	S55°18'W	- 161.390	- 233.078
5	EA	173.16	S02°40'W	- 172.972	- 8.056

Calculate the bearings on which the roadways DC and BC should be driven.

Answer : The length and bearing of line DB are calculated as follows -

No	Line	Length	Bearing	Partial Co - Ordinates		Total Co - Ordinates	
				Lot.	Dep.	Lat.	Dep.
4	DE	283 50	S55°18'W	-161.390	- 233.078	- 161.390	- 233.078
5	EA	173 16	S02°40'W	- 172.972	- 8.056	- 334.362	- 241.134
1	AB	217.71	S59045'E	- 109.676	+ 188.065	- 444.038	- 53.069

$$\text{Bearing of line DB} = \tan^{-1}\frac{-53.069}{-444.038} \quad \text{SW} = \text{S } 6°48'55'' \text{ W.}$$

$$\text{Length of DB} = \sqrt{444.038^2 + 53.069^2} = 447.198 \text{ m.}$$

In triangle DBC :

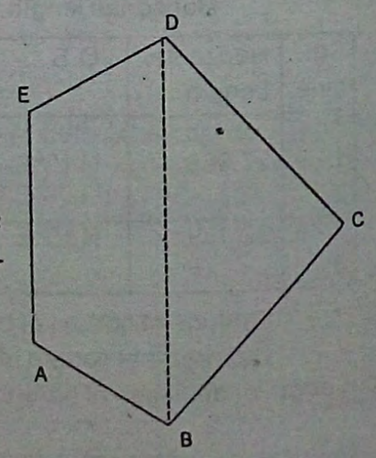
$$\text{Cos BDC} = \frac{BD^2 + CD^2 - BC^2}{2 \times BD \times CD} = \frac{447.198^2 + 375^2 - 318^2}{2 \times 447.198 \times 375} = 0.714037335$$

$$\angle BDC = \text{Cos}^{-1}\ 0.714037335 = 44°26'08''$$

Bearing of DC = Bearing of DB - 44°26'08"

= (180° + 6°48'55") - 44°26'08" = 142°22'47"

= S 37°37'13" E



$$\text{Similarly, Cos DBC} = \frac{BD^2 + CB^2 - DC^2}{2} = \frac{447.198^2 + 318^2 - 375^2}{2 \times 447.198 \times 318}$$

= 0.564257859

∴ Angle DBC = 55°38'57"

Bearing of BC = Bearing BD + 55°38'57"

= 6°48'55 + 55°38'57" = N 62°27'52" E

The roadways BC and DC are to be driven on bearings N 62°27'52" E and S 37°37'13" E.

Examle 47. Following are the notes a survey of a borehole for deviation :

Depth in metres as measured in borehole	*Deviation Angle*	*Direction of deviation Grid Bearing*	*Remarks*
0 - 153.0	0°45'	N 28° E	
153.0 - 360.0	1°30'	N 28° E	
360.0 - 622 5	1°40'	N 20° E	
622.5 - 757 5	2°05'	N 18° E	Top of Seam
757.5 - 761.4	2°05'	N 18° E	Bottom of Seam

From other sources it is known that the full dip of the coal measures is 40 degrees from the horizontal in the direction N 42° E From the information given you are to ascertain the true depth to the bottom of the seam vertically below the surface position of the borehole and the true or stratigraphical thickness of the coal seam.

Answer : To calculate the horizontal and vertical projections of the inclined sections of the borehole, let the horizontal projection = d. Sin θ , and the vertical projection = d. cos θ .

d. Sin θ		d. Cos θ	
153 sin 0°45'	= 2.0027	153 cos 0°45'	= 152.9869
207 sin 1°30'	= 5.4186	207 cos 1°30'	= 206.9291
262.5 sin 1°40'	= 7.6347	262 cos 1°40'	= 262.3889
135. sin 2°05	= 4.9076	135 cos 2°05'	= 134.9107
3.9 sin 2°05'	= 0.1418	3.9 cos 2°05'	= 3.8974

Sum of the verticals = 761.113m

The depth to the bottom of the seam vertically below the surface position of the borehole is therefore 761 113 m.

The apparent dip in the same direction as the last section of the borehole is N 18° E.Full dip of the seam is 40° from horizontal towards N 42° E.

Therefore, $\tan\alpha = \tan\beta \cos\phi$

$= \tan 40° \cos (42° - 18°) = 0.7665556$

$\alpha = \tan^{-1}\ 0.7665556 = 37°28'20''$

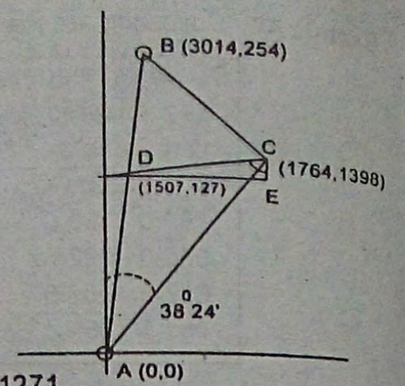
The apparent dip 37°28'20" and that the angle between the line of true thickness of the seam and the line representing the vertical thickness, when we have found to be 3.8974 metres, therefore the true thickness of the seam = 3.8974 cos 37°28'20" = **3.0932 metres.**

Example 48. A straight tunnel is to run between two points A and B whose co - ordinates are given below :

Points	Co - ordinates	
A	0	0
B	3014 N	254 E
C	1764 N	1398 E

It is desired to sink a shaft at D, the middle point AB, but it is impossible to measure along AB directly, so D is to be fixed from C, a third known point.

Calculate : (a) Co -ordinates of D, (b) Length and bearing of CD, (c) Angle ACD, given the bearing of AC as 38°24' East of North.



Answer : (a) Co -ordinates of D :

Latitude of D = (3014 - 0) ÷ 2 = **1507N**

Departure of D = (254 - 0) ÷ 2 = **127 E**

(b) Length of bearing of CD :

CE = 1764 - 1507 = 257m

DE = 1398 - 127 = 1271m

Length of CD = $\sqrt{DE^2 + CE^2}$

$\sqrt{1271^2 + 257^2}$ = **1296.723m**

Bearing of CD = $\tan^{-1}\frac{\text{difference of deps.}}{\text{difference of lats.}} = \tan^{-1}\frac{1271}{257}$ = **S78°34'W**

(c) **Angle** ACD = ∠ECD - ∠ECA = 78°34' - 38°24' = **40°10'**

Example 49 : The following notes refer to a traverse which was made in a mine, starting at the centre of a shaft.

Line	Azimuth	Vernier Angle	Distance
AB	343°16'	—	113.4m
BC		240°12'	123.0m
CD		230°42'	117.0m
DE		264°54'	152.4m

A heading is started at the centre of the sameshaft 71. 1m lower down, and it is driven to rise at 1 in 100 in the direction of N42°30'E. Find the co -ordinates of the point at which the traverse crosses over the heading.

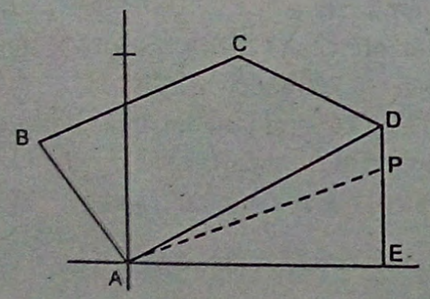
Answer : The figure shows the problem.

Bearing of AB = 360° - 343°16' = N 16°44" W

Bearing of BC = 240°12' -180° - 16°44' = N 43°28' E

Bearing of CD = 230°42' -180° - 43° 28' = S 85°50'

Bearing of DE = 264°54' -180° - 94°10' = S 0°56'E



Line	Bearing	Distancee m	Latitude +	Latitude -	Departure +	Departure -
AB	N 16°44' W	113.4	108.60			32.65
BC	N 43°28'E	123.0	89.27		84.62	
CD	S 85°50'E	117.0		8.50	117.69	
DE	S 00°56'E	152.4		152.38	2.48	
			197.87	160.88	204.79	32.65
			160.88		32.65	
			36.99		172.14	

Length of AE = $\sqrt{36.99^2 + 172.14^2}$ = 176.07m.

Bearing of line AE = N $\tan^{-1}\frac{172.14}{36.99}$ E = N 77°52'21" E

Length of line AD = $\sqrt{\text{total lat}^2 + \text{total dep}^2}$

= $\sqrt{189.37^2 + 169.66^2}$ = 254.25m

Bearing of line AD = N $\tan^{-1}\frac{169.66}{189.37}$ E = N 41°51'28" E

Since the bearing of the new heading is N 42°30'E, the angle A in triangle DAP =(42°30' + 41°51'28") = 0°38'32"

As the bearing of DE is S 00°56'E, and the bearing of AD is N 41°51'28" E, the angle PDA = 42°47'28"

The angle APD is 180° - 42°47'28"-0°38'32" = 136°34' and the supplemantry angle is, of course = 43°26'

Now, $\frac{DP}{SinA} = \frac{AD}{SinP}$: and DP = $\frac{AD\ sinA}{SinP}$

or, DP = $\frac{254.25 Sin\ 0°38'32"}{Sin43°28'} = \frac{2.85}{0.6875}$ = 4.145m.

Latitute of P relative to D = - 4.145 Cos0° 56' = 4.144m

Latitute of P relative to D = 4.145 Sin0°56' = 0.067m

Latitute of P relative to A = 189.370 - 4.144 = 185.255m

Deprature of P relative to A = 169.660 + 0.067 = 169.727m.

Example 50. The lengths and bearings of a closed traverse ABCDE and the latitudes and departures of the known sides are given below. The lengths of two sides BC and CD could not be measured. Calculate the omitted measurements, BC and CD.

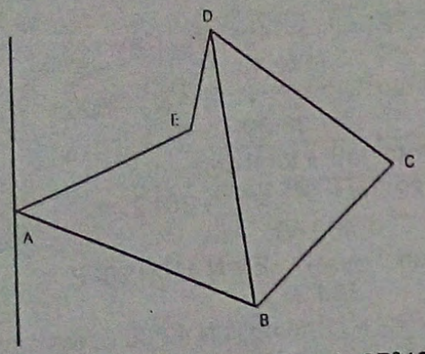
Line	*Length (m)*	*Q.B.*	*Latitude*	*Departure*
AB	730.00	S60°00'E	- 365.00	+ 632.20
BC	X	N 62°18'E	-----	-----
CD	X	N 37°42'W	-----	-----
DE	940.00	S 55°24'W	- 533.77	773.80
EA	575.00	S 2°42'W	- 574.40	- 27.08

Answer : The lines DE, EA and AB are known. DB is the closing line of the polygon DEAB.

Line	*Lats*	*Deps*
DE	- 533.77	- 773.80
EA	- 574.40	- 27.08
AB	- 365.00	+ 632.20
Algebraic sum =	- 1473.17	- 168.68

Closing line BD = $\sqrt{1473.17^2 + 168.68^2}$ = 1482.7956m

Bearing of BD = $\frac{168.68}{1473.17}$ = N6°32'E.



We have, bearing of CD = S 62°18'W, and of CD = N 37°42'E

$\angle BCD = 1800 - (62°18' + 37°42') = 80°$

$\angle DBC = 62°18' + 6°32' = 55°46'$

$\angle BDC = 180° - (80° + 55°48') = 180° - 135°48' = 44°14'$

Using Sine Rule :

$$\textbf{Length of BC} = \frac{BD}{Sin80°} \times \sin 44°14' = \frac{1482.7956}{Sin80°} \times \sin 44°14' = \textbf{1050.328}$$

$$\textbf{Length of CD} = \frac{BD}{Sin\ 80°} \times Sin\ Sin\ 55°46' = \frac{1482.7956}{Sin\ 80°} \times Sin\ 55°46' = \textbf{1244.818m.}$$

Example 51. The centre line of a light railway is to be tangential to each of the following lines :

Line	W.C.B.	Length
AB	0°	—
BC	270°	165m
CD	220°	—

Find the radius of the curve and the tangent lengths.

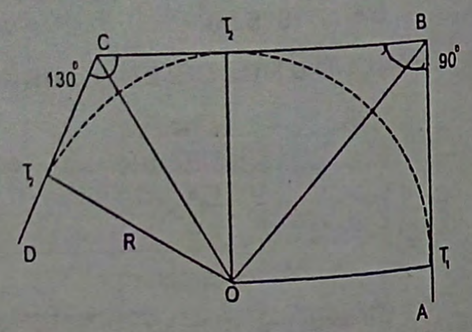
Answer : Let the bisectors of the angles ABC and BCD meet at 0, which is then the centre of the curve. From 0 drawn perpendiculars OT_1, OT_2 and OT_2 to AB, BC, and CD respectively. T_1, T_2 and T_3 are then the tangent points

In $\triangle OBC$, $\angle OBC = 45°$, $OCB = 65°$ and $\angle COB = 180° - 45° - 65° = 70°$ and BC = 165m.

By Sine Rule, we get

$$OC = \frac{165 \sin 45°}{Sin70°} = 124.17m.$$

$$OB = \frac{165 \sin 65°}{Sin70°} = 159.18m.$$



Radius of the curve = $OT_1 = OC \sin 65°$

$= 124.17 \sin 65° = 112.56m.$

Tangent length $BT_1 = BT_2 = 112.56 \tan 45° = 112.56m.$

Tangent length $BT_3 = CT_2 = 112.56 \tan 25° = 52.5m$

Example 52. The co-ordinates of three trig. stations A,B and C are as follows :

A	769m. N	823m.E
B	447m. N	674m.E
C	835m. S	100m.W

Calculate the angles and sides of triangle ABC, the area of the triangle and radius of the circle which passes through vertices of the figure. Check your answer by plotting.

Answer : Length of line AB $= \sqrt{(447 - 769)^2 + (-674 - 823)^2}$

$= 1531.24m.$

Length of line BC $= \sqrt{(-835 - 447)^2 + (100 - 674)^2} = 1404.63m.$

Length of line CA $= \sqrt{\{769 - (-835)\}^2 + \{-100 - 823\}^2} = 1850.61m.$

Bearing of line AB $= \tan^{-1} \frac{-674 - 823}{447 - 769} = S77°52' W$

Bearing of line BC $= \tan^{-1} \frac{-100 + 674}{-835 - 447} = S24°07'E$

Bearing of line CA $= \tan^{-1} \frac{823 + 100}{769 + 835} = N\ 29°55'\ E$

$\angle ABC$ = Bearing of BC - Bearing of BA = S24°07'E - N 77°52'E

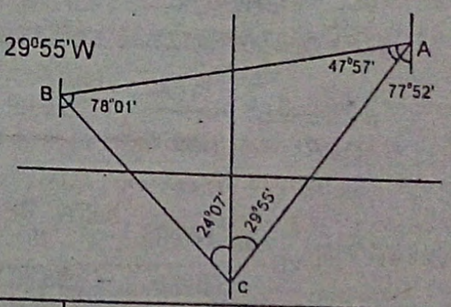
$= 155°53' - 77°52' - 77°52' = 78°01'.$

$\angle BCA$ = Bearing of CA - Bearing CB = N 29°55'E - N24°07'W

$= 54°02',$

$\angle BAC$ = Bearing of AB - Bearing of AC = S 77°52'W - S 29°55'W

$= 47°57'$



Area of Triangle ABC

Stn.	Total Lat.	Total Dep.	Sum of Adj. Lat.	Differnce of Dep.	Double Area +	Double Area −
A	+ 769	+ 823	+ 1216	- 1497		1820.352
B	+ 447	- 674	- 388	+ 574		222.712
C	- 835	- 100	- 66	+ 923		60,918.

Double Area = 2103,982

Area = 1051991 sq.m

Radius of circle $= \frac{1}{2}$ AB Cosec ACB

$= \frac{1}{2} \times 1531.24 \times Cosec\ 54°02'$

$= 765.62 \times Cosec\ 54°02'$

$= \textbf{945.958 m.}$

Example 53. The co-ordinates of three trig. stations A,B and C are :

Stn	*Lat.(M)*	*Dep.(M)*
A	30 N	30 W
B	48 N	86.1 E
C	23.4 S	194.4 E

The angles measured at a colliery triangulation P are :
Angle APB = 57°20' and Angle BPC = 46°00'
Calculate the co - ordinates of station P.

Answer : $\text{Tan AB} = \frac{\text{Departure}}{\text{Latitude}} = \frac{86.1 - (-30)}{48 - 30} = \frac{116.1}{18} = 6.45$

Bearing AB = N ($\tan^{-1}$ 6.45) E = N 81°11'E = Azimuth 81°11'

$\text{Tangent bearing BC} = \frac{149.4 - 86.1}{-23.4 - 48} = \frac{63.3}{71.4} = 0.8865$

Bearing BC = S 41°34'E = Azimuth 138°26'
Angle ABC = 81°11' + (180° - 138°26') = 122°45'

$\text{Length of AB} = \frac{116.1}{\text{Sin } 81°11'} = 117.49\text{m}$

$\text{Length of BC} = \frac{71.4}{\text{Cos } 41°34'} = 95.43\text{m}$

Angle (A + C) = 360° - (122°45' + 57°20' + 46°00') = 133°55'

$\frac{BP}{\text{Sin A}} = \frac{117\ 49}{\text{Sin } 57°20'}$ or $BP = \frac{117.49 \text{ Sin A}}{\text{Sin } 57°20'}$

$\frac{BP}{\text{Sin C}} = \frac{95.43}{\text{Sin } 46°00'}$ and so, $BP = \frac{95.43 \text{ Sin C}}{\text{Sin } 46°00'}$

Therefore $\frac{117.49 \text{ Sin A}}{\text{Sin } 57°20'} = \frac{95.43 \text{ Sin C}}{\text{Sin } 46°00'}$

Thus $\frac{\text{Sin A}}{\text{Sin C}} = \frac{95.43 \text{ Sin } 57°20'}{117.49 \text{ Sin } 46°00'} = 0.950$

But $\angle A = 133°55' - C$ and $\frac{\text{Sin } 133.55 - C}{\text{Sin C}} = 0.950$

$\frac{\text{Sin } 133°55' \text{ Cos C} - \text{Cos } 133°55' \text{ Sin C}}{\text{Sin C}} = 0.950$

Sin 133°55' Cot C = 0.950 + Cos 133°55'

$\text{Cot C} = \frac{0.950 + \text{Cos } 133°55'}{\text{Sin } 133°55'} = \frac{0.950 - \text{Cos } 46°55'}{\text{Sin } 46°05'}$

Therefore ∠C = 70°20'
Angle A = 133°55' - 70°20' = 63°35'
Azimuth Bearing of AP = 81°11' + 63°35' = 144°46'
Bearing AP = S 35°14' E
Angle ABP = 180°00' - (63°35' + 57°20') = 59°05'

In triangle ABP : $\frac{AP}{\text{Sin } 59°05'} = \frac{117.49}{\text{Sin } 57°20'}$

Therefore $AP = \frac{117.49 \text{ Sin } 59°05'}{\text{Sin } 57°20'} = 119.73\text{ m.}$

Partial Co- ordinates of P can be calculated as follows :
Partial Lat = 119.73 Cos 35°14' = 97.80m.S
Partial Dep = 119.73 Sin 35°14' = 69.07m.E
Total Co -ordinates of P are :
Latitude = 30 - 97.80 = - 67.80 m (South)
Departure = - 30 + 69.07 = + 39.07 m (East).

Example 54. The following traverse is run from A to E between which there occur certain obstacles.

Line	*Distence in Metres*	*Bearing*
PQ	133.5	35°15
QR	157.2	345°18
RS	178.5	289°14
ST	124.5	31°30

It is required to peg the point midway between P and T. Calculate the lenth and bearing of a line from R to this midway point.

Answer: To find out bearing of line RM, M being the point between P and T, co-ordinates of M and R are calculated as given below.

Stn. Line	Distance Mts	Bearing	Partial Co-ordinates Lat.	Dep.	Total Co-ordinates. Lat.	Dep.
P					0	0
PQ	133.50	35°15'				
Q			+109.02	+ 77.05	+ 109.02	+ 77.05
QR	157.20	345°18'				
R			+ 152.05	- 39.89	+ 261.07	+ 37.16
RS	178.50	289°14'				
S			+ 58.80	- 168.54	+ 319.87	- 66.38
ST	124.50	31030'				
T			+ 106.54	+ 65.05	+ 426.41	- 66.33

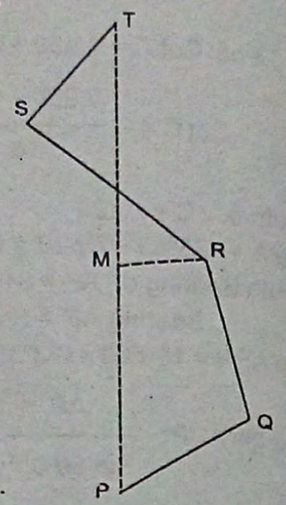
Co- ordintes of Point M which is midway between
Total Co- ordinates of P and T are :

$$\frac{426.41}{2} = 213.205\text{m Total Latitude}$$

and $\frac{-66\ 33}{2} = -33.165\text{m Total Departure.}$

Length of line RM $= \sqrt{\{261.07 - 213.205\}^2 + \{37.16 - (33.165)\}^2}$
$= \mathbf{85.068m.}$

Bearing of RM $= \tan^{-1}\frac{-33.165 - 37.160}{+213.205 - 261.070} = \mathbf{55°45' SW}$



Example 55. It is proposed to connect two points A and B by driving galleries from either point. A traverse is run from A to B and the following bookings are obtained.

Line	*Magnetic Bearing*	*Horizontal Length*	*Gradient*
AC	175°	150m	1 in 5 dip
CD	85°	306m	Level
DE	45°	135m	1 in 8 rise
EB	305°	345m	I in 10 rise

Calculate length, Bearing and Gradient of line AB.

Answer : Assuming original value of station A are : Latitude = O m, and Departure = O m, the Co- ordines of other stations are tabulated below :

Line Stn	Magnetic Bearing	Hor. Length Metre	Partial Co-ordinates		Total Co -ordinates	
			Lat.	Dep.	Lat.	Dep.
A					0	0
AC	175°	150				
C			- 149.429	+ 13.073	- 149.429	+ 13.073
CD	85°	306				
D			+ 26.669	+ 304.835	- 122.760	+ 317.908
DE	45°	135				
E			+ 95 459	+ 95.459	- 27.301	+ 413.367
EB	305°	345				
B			+ 197.884	- 282.607	+ 170.583	+ 130.760

Horizontal Length of AB $= \sqrt{170.583^2 + 130.760^2} = \mathbf{214.934m.}$

Bearing of AB $= \tan^{-1}\frac{+130.760}{+170.583} = 37°28'$

Calculation of Rise and Fall from A to B -

Line	Hor. Length	Gradient	Rise or Fall
AC	150m	1 in 5 dip	- 30m
CD	306m	Level	0 m
DE	135m	1 in 8 Rise	+ 16.87m
EB	345m	1 in 10 Rise	+ 34.50m

Difference of level of A and B = 16.87 + 34.50 - 30 = 21.37m.

Gradient of AB $= \frac{21.37}{214.934} = \mathbf{1\ in\ 10.057}$

The line AB in rising at gradient of 1 in 10.057

Inclined Length of line AB $= \sqrt{214.934^2 + 21.37^2} = \mathbf{215.993}$

Exmple 56. The following are the notes of a traverse survey from station A to station D.
Angle ABC = 118°15'18" and angle BCD = 108°40'42"
Distance AB = 82.8m, BC = 62.361m and CD = 89.976m.
Determine the Co- ordinates of D relative to A, the length of AD and the angle BAD. The line AB is meridian.

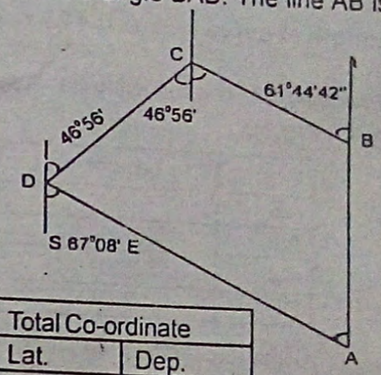
Answer : AB is towards NORTH.

Bearing of BC = 180° + 118°15'18" = 298°15'18" W.C.B.

Q.B. of BC = 360° - 298°15'18" = 61°44'42" NW

Bearing of CD = 108°40'42" - 61°44'42"

= 46°56'00"



Stn. Line	Length m	Q.B.	Partial Co - Ordinate		Total Co-ordinate	
			Lat.	Dep.	Lat.	Dep.
A					0	0
AB	82.800	Due North				
B			+ 82.800	0	82.800	0
BC	62.361	N61°44'42" W				
C			+ 29.521	- 54.930	+ 112.321	- 54.930
CD	89.976	S46°56'00" W				
D			- 61.440	- 65.733	+ 50.881	- 120.663

Length of AD $= \sqrt{50.881^2 + (-120.663)^2} = 130.952\text{m.}$

Bearing of DA $= \tan^{-1}\frac{120.663}{50.881} = S\ 67°08'E$

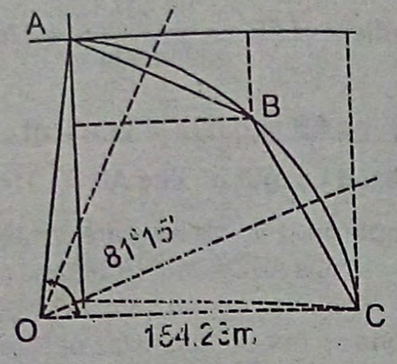
∠BAD = Bearing of AD - Bearing of AB
= N 67°08'W - Due North.
= 292°52' -0 = 292°52' (Exterior angle)

∠BAD (Interior angle) = 360°-292°52' = 67°08'

Example 57. Following are the Co-ordinates of two points B and C in relation to a point.
B. 35.66m. South and 84.12m.East from A.
C. 146m. South and 137.77m East from A.
Plot the positions of the three points to a scale of 1cm = 20m and find by construction the radius of a circular curve passing through the three points A,B and C. Thereafter from the measurements taken from the drawing calculate the length of the curve from A to C.

Answer : The construction of the figure is made by the following steps -

(i) Plotting the points B and C relative to the point A at the intersection of the Co-ordinates.
(ii) Joining the points A and B , and B and C, by straight lines.
(iii) Drawing the perpendicular bisectors of these lines to meet at the point 0. (iv) Drawing the circular curve through the points A , B and C. (v) Measuring the radius of the curve (R=154) and the angle subtended by the curve at the point 0. Angle 0. Angle AOC = 81°15'



Having got the radius of the curve and the central angle, the length of the curve from A to C in calculated as follows :

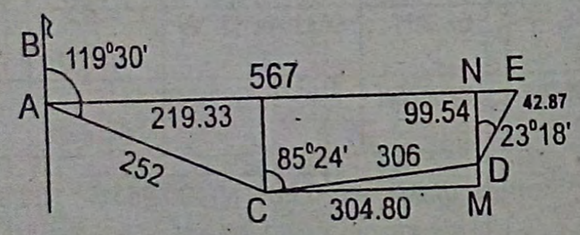
Length of curve = $2 \times \pi \times 154.23 \times \frac{81°15'}{360°}$ = **218.74m.**

Example 58. The following traverse is run to set out a line AE 567.0 m long at right angles to a given line AB :

Line.	Length in m.	Bearing	Lat. in m.	Dep. in m.
AB	-	360°	-	-
AC	252.0	119°30'	-124.08	+ 219.33
CD	306.0	85°24'	+ 24.54	+ 304.80
DE	?	?	-	-

Calculate the required length and bearing of DE.

Answer : The bearing of AB is 360°, the bearing of AE = 90°, and that of EA = 270°.
Latitude of EA = 0, and Departure of EA = - 567.0m.
Now, - 124.08 + 24.54 + L + 0 = 0
L = 99.54m.
and 219.33 + 304.80 + D - 567.00 = 0
D = 42.87 m.



Bearing DE = $\tan^{-1} \frac{42.87}{99.54}$ = 23°18'

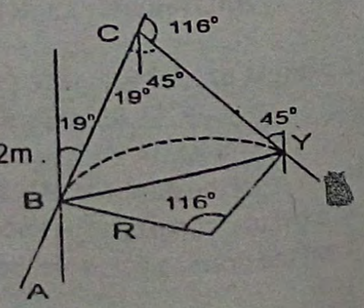
Length of DE = $99.54 \times \frac{1}{\text{Cos } 23°18'}$ = **108.378 m**

Example 59. Two roads are to be joined by a smooth curve. The directions and Co -ordinates of the road turning points are as follows : Direction of the roads AB in N 19°E and Latitude and XY N45°W. The Co- ordinates of the tangent point B = 3080 m Latitude and 1590m Departure. The Co- ordinates of the tangent point Y = 3150m Latitude and 1590m Departure. Find out the radius of the curve

Answer : Length of the long chord By

$= \sqrt{(1590 - 1500)^2 + (3150 - 3080)^2} = \sqrt{90^2 + 70^2} = 114.02$m .

YCB = 19° + 45° = 64°
Angle at the centre = 180° - 64° = 116°



Length of long chord = 2 R Sin $\frac{116°}{2}$ = 114.02 m

Radius, $R = \frac{114.02}{2 \sin 58°}$ = **67.225m.**

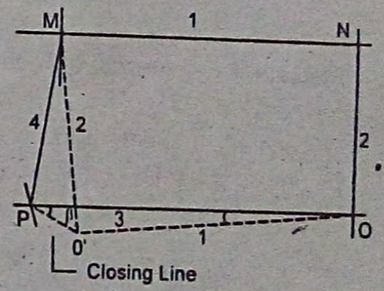
Example 60. A four sided traverse MNOP, has the following lengths and bearing :

Line	Length (m)	Bearing :
MN	500	Roughly East
NO	245	178°
OP	Not obtained	270°
PM	216	10°

Find the exact bearing of the line MN.

Answer : Traverse MNOP shows in continious line. The affected lines MN and OP are not adjacent. To bring MN adjacent to OP, draw a line MO' , equal and parallel to NO.The line OO' is thus equal and parallel to NM and is adjacent to OP. The line O'P becomes the closing line of the traverse MO'P. The latitude and departure of O'P can be calculated as usual. The calculations are shown below :

Line	Lat.	Dep.
PM	+ 212.71	+ 37.51
MO'	- 239.64	+ 50.94
(No)		
sum	- 26.93	+ 88.45



Latitude of O'P = + 26.93
Departure of O'P = - 88.45
The bearing (θ) of O' P is given by-

$\tan \theta = \frac{88.45}{26.93}$ $\quad \therefore \theta = 73°4'$

∴ Bearing of O'P = N 73°4' W = 360° - 73°4' = 286°56'
∠ β = (270° -180°) - 73°4' = 16°56'
Length of O'P = Lat. x sec = 26.93 x sec 73°4' = 92.47m.

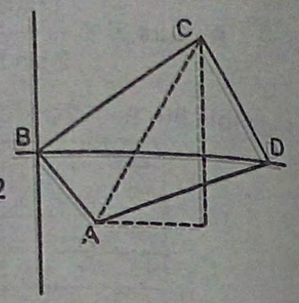
From triangle OPO', $\frac{OO'}{\text{Sin } \beta} = \frac{PO'}{\text{Sin } \alpha}$

$\text{Sin } \alpha = \frac{PO'}{OO'} \times \text{Sin } \beta = \frac{92.47}{500} \times \text{sin } 16°56'$

∝ = 3°55'
Bearing of NM = Bearing of 00' = 270° - 3°55' = 266°5'
Bearing of MN = 266°5' -180° = 86°05'.

Exaple 61. Four stations A, B, C and D form a quadrilateral figure , station D being due East of B with station C North of A and A south of line BD. The distances between the stations are AB= 586.8m, BC = 1008m, CD = 867.6m, DA = 1215m and BD =1454.4m Calculate the distance AC.

Answer : Figure shows the relative positions of the stations. Calculate the angles CBD and BDA.



In triangle BCD, the semi - perimeters (s) of the triangle

$$= \frac{1}{2}(867.6 + 1454.4 + 1008) = 1665\text{m}.$$

$$\text{Sin}\ \frac{B}{2} = \sqrt{\frac{(s-c)(s-d)}{cd}} = \sqrt{\frac{(1665-1454.4)(1665-1008)}{1454.4 \times 1008}} = 0.3072$$

Therefore $\frac{\angle B}{2} = 17°53'$ and $CBD = 35°46'$

The bearing of the line BC will then be $90° - 35°46' = 54°14'$ NE

Departure of C relative to B = 1008 sin 54°14' = 817.895m East

Latitude of C relative to B = 1008 cos 54°14' = 589.161m North.

In triangle DBA, semiperiters (s) = ½ (1454.4 + 1215 + 568.8) = 1619.1

$$\text{Sin}\ \frac{B}{2} = \sqrt{\frac{(s-a)(s-d)}{ad}} = \sqrt{\frac{(1619.5-1454.4)(1619.5-568.8)}{1454.4 \times 568.8}} = 0.4579$$

Therefore $= \frac{\angle B}{2}$ 27°15' and DBA = 54°30'

The bearing of line BA will then be $(90° - 54°30') = 35°30'$ SE

Departure of A relative to B = 568.8 sin 35°30' = 330.304m East.

Latitude of A relative to B = 568.8 Cos 35°30' = 463.069m South.

Length of AC = $\sqrt{(\text{diff. of deps})^2 + (\text{diff. of lats})^2}$

= $\sqrt{(817.895 - 330.304)^2 + (589.161 + 463.069)^2}$

= $\sqrt{487.591^2 + 1052.23^2}$

= **1159.712m.**

Example 62. Stations P,Q and R in the working of a seam are to be joined by a circular curve. The Co -ordinate in metres, of P and Q in relation to O are -

Station	*Latitude*	*Departure*
Q	N 57.60	E 21.50
R	S 75.00	E 80.00

Calculate the radius of the circular curve to pass through the stations P,Q and R.

Answer : Tan $\theta_1 = \frac{21.50}{57.60}$ $\therefore\ \theta = 20°28'$

Length of QR = $\sqrt{(57.60 + 75.00)^2 + (80.00 - 21.50)^2}$ = 144.93m.

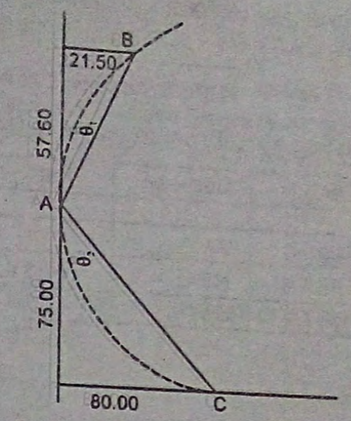
Tan $\theta_2 = \frac{80.00}{75.00}$ $\theta_2 = 46°50'$

Included angle QPR

= 180° - (20°28' + 46°50') = 112°42'

Now $\frac{QR}{\text{Sin QPR}} = 2R$

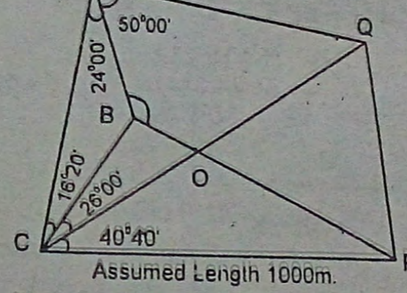
Then $R = \frac{144.93}{2 \sin 112°42'} = 78.55\text{m}.$



Example 63. The Co - ordinates, in metres, in relation to the point of origin of the two triangulation stations Q and P, shown in the figure which is not to scale, are as follows :

	Latitude	*Departure*
Q.....	+ 3200	- 600
P.....	+2046	+ 100

The figure also shows part of a surface triangulation with certain angles obtained by theodolite observations at the stations marked A, B and C, Calculate the Co -ordinates of stations B.



Given Angles	*Deduced Angles*
QCP = 40°40'	AQC = 63°40'
BCQ = 26°.00'	BPC = 33°20'
ACB = 16°20'	ABC = 139°40'
BAC = 24°00	BOQ = Q + P = 106°
QAB = 50°00'	
CBP = 80°00'	

Answer : Assumed CP = 1000m and assume bearing CP is due east.

In triangle BCP, BC = $\frac{CP \text{ Sin CPB}}{\text{Sin PBC}} = \frac{1000 \text{ Sin } 33°20'}{\text{Sin } 80°00'} = 557.8\text{m}$

In traingle BCA, AC = $\frac{BC \text{ Sin CBA}}{\text{Sin BAC}} = \frac{557.8 \text{ Sin } 139°40'}{\text{Sin } 24°00'} = 887.9\text{m}$

In traingle QCA, QA = $\frac{AC \text{ Sin ACQ}}{\text{Sin CQA}} = \frac{887.9 \text{ Sin } (16°20' + 26°00')}{\text{Sin } 63°40'} = 667.2\text{m}.$

Assumed azimuth CP = 90°00'

Deduct angle ACP = 83°00'

7°00' Assumed azimuth CA

Add 180°00' (assumed azimuth CA plus)

187°00'

Deduct angle QAC = 74°00'

113°00' Assumed azimuth AQ

Now calculating arbitary co-ordinates of Q in relation to P.

Line	Length	Bearing	Lat.	Dep.	Total Lat.	Total Dep.	Stn.
					0.0	0.0	P
PC	1000 0	W	-	-1000.0	0.0	-1000.0	C
CA	887 9	N7°E	+ 881.3	+108.2	881.3	-891.8	A
AQ	667 2	S67°2	- 260.7	+ 614.2	620.6	-277.6	Q
			620.6	1000.0			
				722.4			
				277.6			

Arbitrary length QP = $\sqrt{(277.6)^2 + (620.6)^2}$ = 679.9m.

Arbitrary bearing PQ = $\tan^{-1} N \frac{277.6}{620.6} W$ = N24°06' W

Angle CPQ = (335°54' -270°00') = 65°54'
Angle BPC = (65°54' -33°20') = 32°34'

True Bearing PQ = $\tan^{-1} N \frac{709}{1154} W$ = N 31°34' W

Therefore, true bearing PB =32°34' + 31°34' = N 64°08' W

True Length PQ = $\sqrt{(709)^2 + (1154)^2} = \sqrt{1834400}$ = 1354.4m.

Therefore, true length PC = 1000 x $\frac{1354.4}{679.9}$ = 1992.2m.

In triangle BCP

$BP = \frac{CP \sin BCP}{\sin PBC} = \frac{1992.2 \sin (40°40' + 26°00')}{\sin 80°00'}$ = 1857.5m.

Latitude of B with P as zero = 1857.5 cos 64°08' = N 810.4m.
Departure of B with P as zero = 1857.5 sin 64°08' = W 1671.4m.
Therefore, **reduced latitude of B = + 2046 + 810.4 = + 2856.4m.**
reduced departure of B = + 109 - 1671.4 = - 1562.4m

Example 64. The following are the notes for a theodolite traverse made in a roadway in a certain coal seam :

Line	Horizontal Angle	Azimuth	Dist. m.
AB	—	88°28'	440.0
BC	170°15'	78°43'	393.6
CD	205°47'	104°30'	181.0
DE	171°31'	96°04'	232.1

The levels in metre above a certain datum, taken of the floor of the roadway, are -
A = 70.7, B =73.1, C = 75.0, D = 76.9, E = 78.2

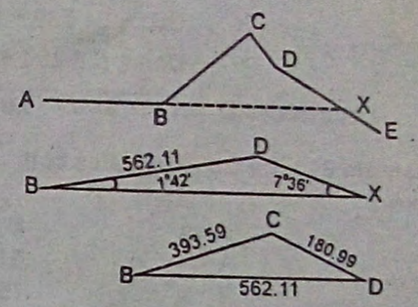
A new roadway is to be driven from the point B to the point X where the line AB extended intersects the line DE, and this roadway is to have uniform of gradient from B to X. Calculate the distance that the point X will be from D and the length and gradient of the new roadway BX.

Answer : Horizontal distance BC = $\sqrt{393.6^2 - (75 - 73.1)^2}$ = 393.59m.
Horizontal distance CD = $\sqrt{181^2 - (76.9 - 75.)^2}$ = 180.99m.
The reduction of right angled triangles, the hypotenuse being the measured length , the vertical being the diference in level.
In triangle ABC

$BD^2 = (393.59)^2 + (180.99)^2 - 2(393.59)(180.99)\cos 154°13'$

BD = 562.11m

$\frac{562.11}{\sin 25°47'} = \frac{180.99}{\sin B}$



Therefore ∠B = 8°03'

Azimuth of BD = 78°43' + 8°03' = 86°46'
In triangle BDX

∠DBX = 1°42' , ∠BDX = 17°42' , ∠DXB = 7°36'

Now $\frac{562.11}{\text{Sin } 7°36'} = \frac{BX}{\text{Sin } 9°18'}$ ∴ BX = 686.84 m.

$\frac{686\ 84}{\text{Sin } 9°18'} = \frac{DX}{\text{Sin } 1°42'}$ ∴ DX = 126.09 m.

Distance of X from D = 126.09m
DE rises 1.3 in 232.1

Therefore DX rises $\frac{126.09 \times 1.3}{232.1}$ = 0.706

Level of X = 76.9 + 0.706 = 77.606m and since level of B = 73.1m, rise B to X = 4.506m.
Gradient of 4 5 in 686.84 = 1 in 152.6 along BX.

Example 65. The following are the notes of an underground traverse.

Line	*Quadrant Bearing*	*Hor.Dist.*
AB	N 35°06' E	90m
BC	N 03°18'W	117m
CD	S 72°42' E	31.5m

The Co - Odinates of A based on a local origin are 94.56m S and 38.28m W. Calculate the Co -Ordinates of the stations B,C and D. It is proposed to drive a new roadway in a direct line between the points A and D. Calculate the quadrant bearing and length of the line AD.

Answer : Co -Ordinates of a based on A local origin are given. The total Co -Ordinates of the stations A, B, C and D are calculated and tabulated below.

Stn.	Line	Quadrant Bearing	Dist. m.	Partial Co -Ordinates		Total Co - Ordinates	
				Lat.	Dep.	Lat.	Dep.
A				- 94.560	- 38.280	- 94.560	-38.280
	AB	N 35°06' E	90				
B				+ 73.633	+51.750	-20.927	+13.470
	BC	N 03°18' W	117				
C				+116.805	- 6.735	+ 95.878	+ 6.735
	CD	S 72°42' E	31.5				
D				- 9.367	+ 30.075	+ 86.511	+ 36.810

Bearing of AD = $\tan^{-1} \dfrac{(-38.280) - (36.810)}{(-94.560) - (86.511)} = \dfrac{-75.09}{-181.07}$ = **22°31' NE.**

Lat. +(94.560 + 86.511) = 181.071

Dep.+ (38.280 + 36.810) = 75.090

Length of AD = $\sqrt{181.071^2 + 75.090^2}$ = **196.02m.**

Example 66. The Co - Oridnates of three points in meters to a local origin are respectively as follows -

Stns.	*Latitude*	*Departure*
A	619.50	64.02
B	761.10	1389.45
C	302.82	951.63

A and B are the points on an underground roadway. Another roadway is to be driven in a direction of N 10°00' W from C to intersect AB. Determine the length of CD

Answer : Length of AC = $\sqrt{(619.5 - 302.82)^2 + (951.63 - 64.2)^2}$

= $\sqrt{316.68^2 + 887.61^2}$ = 942.41m

Bearing AC = S $\tan^{-1} \dfrac{887.61}{316.68}$ E = S 70°21'E

Bearing AB = N $\tan^{-1} \dfrac{1389.45 - 64.02}{761.1 - 619.5}$ E = N83°54' E

Angle ACD = 70°21' - 10°00' 60°21'

Angle DAC = (90°-83°54') + (90°-70°21') = 25°45'

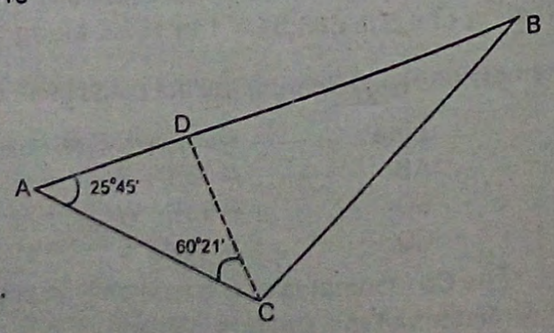
Angle ADC = 180° - (60°21' + 25°45') = 93°54'

Now $\dfrac{AC}{\text{Sin ADC}} = \dfrac{CD}{\text{Sin CAD}}$

$CD = \dfrac{AC \times \text{Sin CAD}}{\text{Sin ADC}}$

$= \dfrac{942.41 \times \text{Sin } 25°45'}{\text{Sin } 93°54'}$ = **410.376m.**

Length of CD = 410.376m.



Example 67. Determine the volume of common excavation at a junction of two roadways both of rectangular cross-secton 3m high and 2.7m wide. One roadway is straight and tangential to a current roadway of 30m radius.

Answer : The two roadways are shown in a figure, being tangential to the curved roadways of 30m radius. The shaded portion shows the area of common excavatoin.

Width of road = 2.7m.and Radius of of road curve = 30m.

Therefore, radius of outer curve of the road = $30 + \dfrac{2.7}{2}$ = 31.35m.

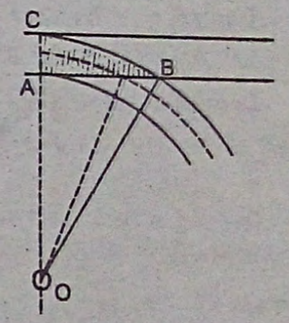
From the sketch -

AB = $\sqrt{31.35^2 - 28.65^2}$ = 12.73m.

Now, $\dfrac{OB}{\text{Sin } \angle OAB} = \dfrac{AB}{\text{Sin } \angle AOB}$

$\text{Sin } \angle AOB = \dfrac{AB}{OB} \times \text{Sin } \angle OAB$

$= \dfrac{12.73}{31.35} \times \text{Sin } 90^0 = \dfrac{12.73}{31.35} \times 1 = 0.406$



$\angle AOB = 23°57'$.

Area of sector OCB = $\dfrac{\angle AOB}{360^0} \times \pi (31.35)^2$

$= \dfrac{23.95 \times 3.1416 \times 31.35^2}{360}$ = 205.48 sq.m.

Area of triangle AOB = ½ OA x AB = ½ x 28.65 x 12.73 = 182.36 sq.m.

Therefore, area of common excavtion = 205.48 - 182.36 = 13.12 sq.m.

As the sections of the roadway is rectangular and the height of road is 3m,

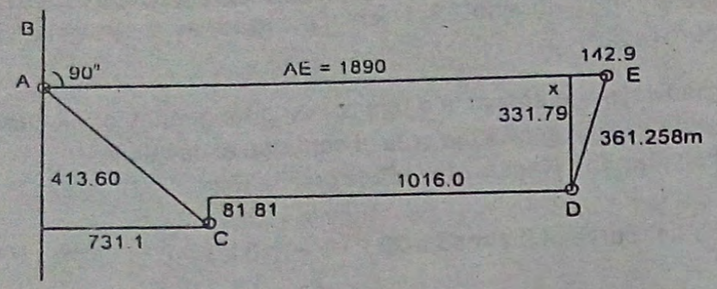
Volume of common excavation = 13.12 x 3 = **39.36 cubic m**

Example 68. The following traverse is run to set out a line AE 1890m long at right angles to a given line AB :

Line	Length (m)	Bearing	Latitude (m)	Departure (m)
AB	–	360°	–	–
AC	840.00	119°30'	- 413.60	+ 731.10
CD	1020.00	85°24'	+ 81.81	+ 1016.00
DE	?	?	–	–

Calculate the length and bearing of DE.

Answer : The figure shows the given points in the question. It shows the simple geometrical figures lying between them



AX = 731.1 + 1016.0 = 1747.1m.

Dep.XE = 1890 - 1747.1 = + 142.9m.

Lat. DX = 413.60 -81.81 = + 331.79m.

Length of DE = $\sqrt{Dep^2 + Lat^2}$

$= \sqrt{142.9^2 + 331.79^2}$ = **361.258m.**

Bearing of DE = $\tan^{-1}\frac{142.9}{331.79}$ = 23°18' NE.

Example 69. Two straight lines PQ and QR on the centre-line of a proposed road on a rocky headland are to be connected by a circular curve of 180m radius. From the traverse notes,it is found that if the bearing of PQ is assumed to be N0°0' E the bearing of QR will be N48°20'E, while if P be taken as the origin in Co -Ordinates, the latitude and departure of R will be + 217.5 m and 109.5m respectively. Determine the distance of the tangent points of the curve from the stations P and R.

Answer : In figure, ∠R'QR = 48°20'

Latitude of R = PR' = 217.5m

Departure of R = PR " = 109.5m

Length of PR = $\sqrt{217.5^2 + 109.5^2}$ = 243.508m.

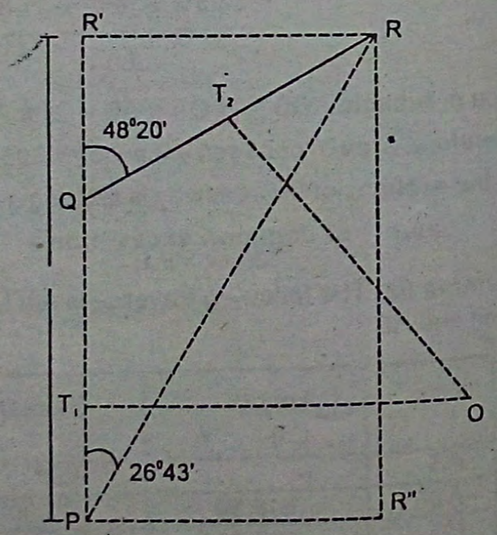
Bearing of PR = $\tan^{-1}\frac{D}{L}$

$= \tan^{-1}\frac{109.5}{217.5}$ = N 26°43'

∠QPR = 26°43'

∠RQP = 180°-48°20' = 131°40'

∠QRP = 48°20' - 26°43' = 21°37'



From ΔQPR : $QP = \frac{PR}{\text{Sin } 48°20'} \times \text{Sin } 21°37'$

$= 243.508 \times \frac{\text{Sin } 21°37'}{\text{Sin } 48°20'} = 120.086$

$QR = \frac{PR}{\text{Sin } 48°20'} \times \text{Sin } 26°43'$

$= 243\ 508 \times \frac{\text{Sin } 26°43'}{\text{Sin } 48°20'} = 146.549\text{m}.$

Circular curve tangemt is given by -

$T = QT_1 = QT_2 = R \tan \frac{48°20'}{2} = 180 \times \tan \frac{48°20'}{2} = 80.769\text{m}$

Distance PT_1 = QP - QT_1 = 120.086 - 80.769 = **39.317m.**

Distance RT_2 = QR - QT_2 = 146 549 -80 769 = **65.780m.**

Example 70. From a point Q a line QR is to to be set out parallel to a given line MN, and such that MNR is a right angle, the points Q and R being invisible from M and N. This is done by running the following traverse :

Line	*Length (m)*	*Bearing*	*Latitude (m)*	*Departure (m)*
NM	-	360°	-	-
NO	96.0	35°15'	+ 78.39	+ 55.38
OP	144.0	115°40'	- 62.37	+ 129.78
PQ	87.0	42°30'	+ 64.14	+ 58.77

Calculate the necessary lengths and bearings of QR and RN.

Answer : Bearing of NM = 360° ∴ Bearing of MN = 180°

QR is parallel to MN, the bearing of QR = 180°

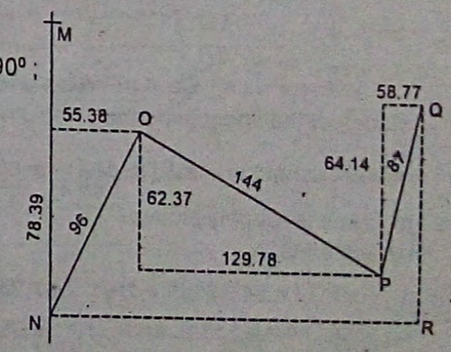
The angle MNR being 90°, the bearing of NR = 90° ; and that RN = 270°.

Lat. = 78.39 - 62.37 + 64.14 = 80.16m

Dep. = 55.38 + 129.78 + 58.77 = 243.93m

Length of QR = 80.16m.

Length of RN = 243.93m.



Example 71. The Co -Ordinates in metres of Two points A and B are -

A 1000 N 180E ; B 870N 900E.

The point A is on the outer fence of a circular railway enclousere, which is 100m wide, and the point B is on the inner fence. Given that the radius of the centre line of the railway is 660m, find the angles of the triangle AOB, where O is the centre of the circle on the centre line.

Answer : From the gven Co -Ordinates -

Length of AB = $\sqrt{(1000 - 870)^2 + (900 - 180)^2}$ = 731.64m

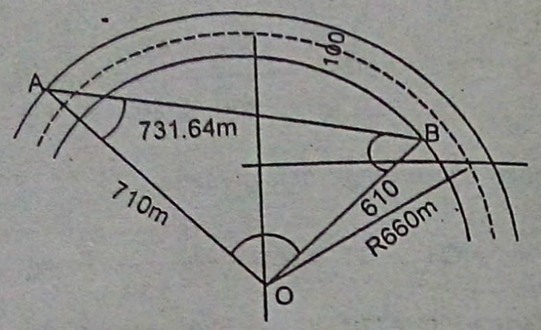
In triangle AOB, AB = 731.64m, AO = 660 + 50 = 710 m

and BO = 660 - 50 = 610m.

$AB^2 = AO^2 + BO^2 - 2\,(AO \times BO)\cos \angle AOB$

or, $\dfrac{AO^2 + BO^2 - AB^2}{2\,(AO \times BO)} = \cos \angle AOB$

or, $\dfrac{710^2 + 610^2 - 731.64^2}{2 \times 710 \times 610} = \cos \angle AOB$



Angle AOB = 66°49'

Now, $\dfrac{AB}{\text{Sin } \angle AOB} = \dfrac{BO}{\text{Sin OAB}}$

$\text{Sin } \angle OAB = \dfrac{BO \times \text{Sin AOB}}{AB} = \dfrac{610 \times \text{Sin } 66°49'}{731.64} = 0.766418505$

∠OAB = 50°02'

$\text{Sin } \angle OBA = \dfrac{AO \text{ Sin } 50°02'}{OB} = \dfrac{710 \times \text{Sin } 50°02'}{610} = 0.892060611$

∠OBA = 63°08'

Question 72. The following observations were obtained in making a correlation survey to two shaft wires J and H :

Line	*Bearing*	*Distance*
FG	N 84°33'30" E	47.138m
GJ	S 2° 51'00" E	89.253m
FH	S 5° 53' 00 "E	90.028m

Without using Co -Ordinates and without plotting the survey, calculate the bearing and distance of the line joining the wires J and H.

Answer : Length of line JH and its bearing can be found out by solving the triangles trigonometrically.

The figure is roughly plotted.

In triangle FGJ :

$\angle FGJ = 84°33'30" + 2°51' = 87°24'30"$

FG = 47.138m. and GJ = 89.253m.

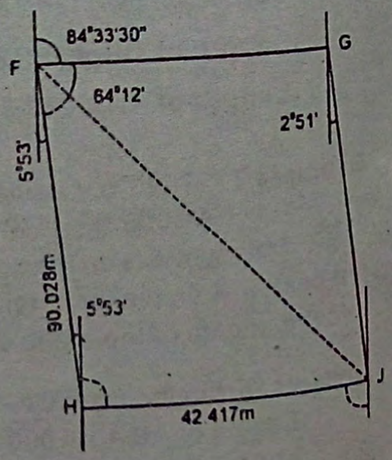
By cosing Rule.

$FJ = \sqrt{47.138^2 + 89.253^2 - 2 \times 47.138 \times 89.253 \times \text{Cos } 87°24'30"}$

= 99.033m.

By Sine Rule,

$\dfrac{GJ}{\text{Sin GFJ}} = \dfrac{FJ}{\text{Sin FGJ}}$ $\quad \therefore \text{Sin GFJ} = \dfrac{GJ \times \text{Sin FGJ}}{FJ}$



$\text{Sin GFJ} = \dfrac{89.253 \times \text{Sin } 87°24'30"}{90.033} = 0.9003232$

$\angle GFJ = 64°12'$

$\angle HFJ = 180° - (84°33'30" + 64°12' + 5°53') = 25°21'30"$

$JH = \sqrt{FJ^2 + FH^2 - 2FH \times FJ \times \text{Cos } 25°21'30"}$

$= \sqrt{99.033^2 + 90.028^2 - 2 \times 99.033 \times 90.028 \times \text{Cos } 25°21'30"}$

= 42.417 m.

From triangle FHJ,

$\dfrac{HJ}{\text{Sin HFJ}} = \dfrac{FJ}{\text{Sin FHJ}}$ or $\dfrac{42.417}{\text{Sin } 25°21'30"} = \dfrac{99.033}{\text{Sin } \angle FHJ}$

$\therefore \quad \text{Sin } \angle FHJ = \dfrac{99.033 \times \text{Sin } 25°21'30"}{42.417} = 0.9999213$

$\angle FHJ = 89°16'\ 53.4$

Bearing of HJ = 89°16' 53.4 - 5°53' = 83°23' 53.4

Therefore, **Bearing of JH = S 83°23'53.4 W.**

Example 73. Given the following latitudes and departures of traverse ABCDE, the bearing of AB and EA having been omitted :

Line	*Latitude*	*Departure*	*Length*
AB	-	-	1970m
BC	+ 841.11	+ 336.71	
CD	+ 877.18	- 311.74	
DE	- 700.60	- 728.88	
EA	-	-	1181m

Determine the bearings of AB and EA.

Answer : To get the latitudes and departures of the closing line EB :

Line	*Lat.*	*Dep.*
BC	+ 841.11	+336.71
CD	+ 877.18	- 311.74
DE	- 700.60	- 728.88
Total	+ 1017.69	- 703.91

The length of the closing line EB = $\sqrt{1017.69^2 + 703.91^2}$ = 1237m

Bearing of the closing line EB = tan R.B. = $\frac{703.91}{1017.69}$

∴ Reduced Bearing of EB = S 34°40'E.

When three sides of a triangle are known

$$\text{Cos A} = \frac{b^2 + e^2 - a2}{2be} = \frac{1181^2 + 19702^2 - 1237^2}{2x\ 1181\ x\ 1970}$$

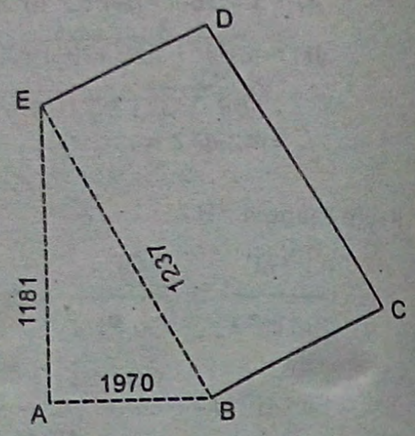
∠A = 36°24'

$$\text{Cos B} = \frac{a^2 + e^2 - b^2}{2\ ae} = \frac{1237^2 + 1970^2 - 1181^2}{2\ x\ 1237\ x\ 1970}$$

∠B = 34°30'

$$\text{Cos E} = \frac{a^2 + b^2 - e^2}{2ab} = \frac{1237^2 + 1181^2 - 1970^2}{2x\ 1237x1181}$$

∠E = 109°06'.



Bearing of line EB = S 34°40'E = 145°20' ∴ BE = 325°20'

∠EBA = 34°30' Bearing BA = 320°20' - 34°30' = 290°50'

Hence bearing of AB = S 69°10'E, and ∠A = 36°24'

∴ **Bearing of EA = 180° - (69°10'+36°24') = S 74°26' W**

Example 74 : The consecutive Co -Ordinates of a closed traverse ABCD are give below in metres. Compute the area of the traverse by departures and total latitude method.

Line	Latitudes	Departures
AB	+ 110	+ 6
BC	+ 17	+ 251
CD	- 127	+ 6
DA	0	- 263

Answer : The calculation are tabulated as follows -

Line	Latitude (L)	Departure (D)	Stn.	Total Lats. L	Algebraic sum of adjoining Departure	Double Area
1	2	3	4	5	6	7
AB	+110	+6	B	+ 110	+257	+28270
BC	+17	+251	C	+127	+257	+32639
CD	-127	+6	D	0	-257	0
DA	0	-263	A	0	-257	0
					Sum	60909

∴ **Area** = $\frac{1}{2}$ x 60909 = 30454.5 sq.m.

= 3.04545 Hectares.

Example 75 : Two roads intersect at the point A. Determine the radius of simple circular curve so as to touch three lines BA, CA and MN. Given BMN = 120°, CNM = 118° and AM = 10 chains, 30 metre in used.

Answer : Intersection angle = •

$180^0 - [(180^0 - \theta_1) + (180^0 - \theta_2)]$

$180^0 - [(180^0 - 120^0) + (180^0 - 118^0)] = 58^0$

Deflection angle = $\phi = 180^0 - 58^0 = 122^0$

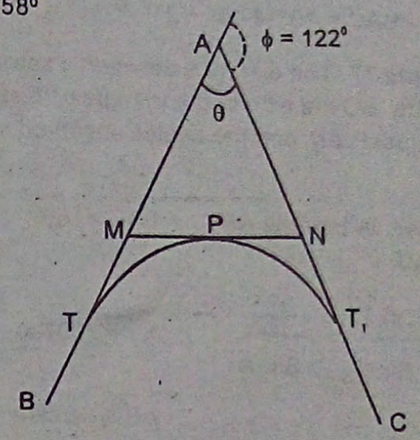
MN = 10 chains.

$$\textbf{Radius} = \frac{AM \times \tan\frac{\theta_1}{2} \times \tan\frac{\theta}{2}}{\tan\frac{\theta_1}{2} - \tan\frac{\theta}{2}}$$

$$= \frac{10 \times \tan\frac{120^0}{2} \times \tan\frac{58^0}{2}}{\tan\frac{120^0}{2} - \tan\frac{58^0}{2}}$$

$$= \frac{10 \times \tan 60^0 \times \tan 29^0}{\tan 60^0 - \tan 29^0}$$

= 244.5m.



Example 76. The distance between satellite station E and the corresponding principal station A is 10m. BC the side of the triangle ABC is 2000m long. The angle ABC and ACB are 60° and 50° respectively and the angles observed at B are ∠AEB =90°, ∠BEC = 70°14'45". Find the angle BAC.

Answer : In figure given -

∠ 70°14'45m" + β = ∠ BAC + y

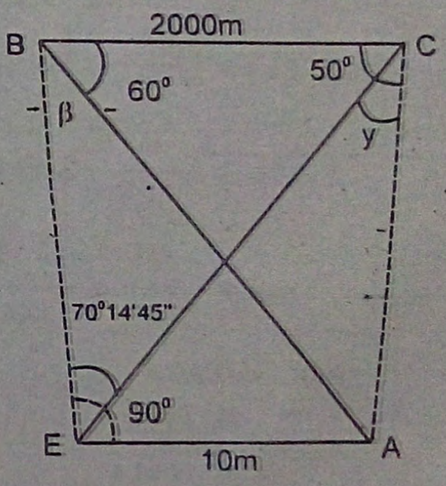
∴ ∠ BAC = 70°14'45" + B -Y

$$\frac{\text{Sin}\,\beta}{10} = \frac{\text{Sin}90^0}{BA}$$

$$\frac{BA}{\text{Sin}\,50^0} = \frac{2000}{\text{Sin}\,70^0} \quad \therefore BA = 1630.41m.$$

$$\frac{CA}{\text{Sin}60^0} = \frac{2000}{\text{Sin}\,70^0} \quad \therefore CA = 1843m.$$

$$\frac{\text{Sin}\,\beta}{EA} = \frac{\text{Sin}\,90^0}{1631} \quad \therefore \beta = \text{Sin}^{-1}\frac{10 \times \text{Sin}90^0}{1631} = 21'.$$



$$\frac{\text{Sin } y}{10} = \frac{\text{Sin}19^0 45'15''}{1843} \quad \therefore Y = \text{Sin}^{-1} \frac{10 \times \text{Sin } 19^0 45'15'}{1834} = 6'18''$$

$\angle BAC = 70^0 14'45'' + 21' - 6'18'' = 70^0 29'27''$

Example 77. The distance between a satellite station E and the corresponding principal station A is 11m. BC the side of the triangle ABC is 2007m Long. The angles ABC and ACB are 63^0 and 54^0 respectively, and the angles observed at E are AEB = 94^0 and BEC = 64^0. Find the angle BAC

Answer: In triangle ABC, $\angle BAC = 180^0 - (63^0 + 54^0) = 63^0$ for approximate computation of side AC.

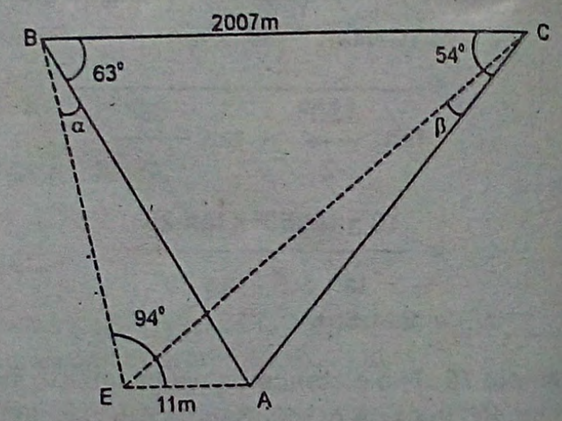
$$\frac{AB}{\text{Sin } C} = \frac{BC}{\text{Sin } A} \quad \therefore AB = 2007 \times \frac{\text{Sin } 54^0}{\text{Sin } 63^0}$$

$$\therefore AB = 1822m.$$

$$\therefore BAC = 180^0 - (63^0 + 54^0) = 63^0$$

Since triangle ABC is isosceles triangle,

$$\therefore BC = AC = 2007m.$$



In ΔBEA, $\frac{\text{Sin}\alpha}{11} = \frac{\text{Sin } 94^0}{AB}$

$$\text{Sin}\alpha = \frac{11 \times \text{Sin } 94^0}{1822} \quad \therefore \alpha'' = 20'4.5''$$

In ΔECA, $\text{Sin}\beta = \frac{EA}{AC} \times \text{Sin } CEA$

$$= \frac{11}{2007} \times \text{Sin } (94^0 - 64^0)$$

$$\beta'' = \frac{11}{2007} \times \frac{\text{Sin } 30^0}{0.000005} = 9'08''$$

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$$\angle BAC = \angle BAC + \alpha'' - \beta''$$

$$= 64^0 + (20'\,4.5'') - (9'08'')$$

$$= 64^0 10'56.4''.$$

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